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Status of Advanced Waste Treatment

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STATUS OF ADVANCED WASTE TREATMENT

I. J. Kugelman

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Environmental Protection Agency
National Environmental Research Center
Advanced Waste Treatment Research Laboratory
Cincinnati, Ohio

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I. J. Kugelman*

INTRODUCTION

The Advanced Waste Treatment Program of the U.S. Environmental Protection Agency was instituted over a decade ago at what is now the National Environmental Research Center in Cincinnati. The charter of this program was and still is to upgrade the state of the art of wastewater treatment technology. Program impetus was provided by recent upgrading of water quality criteria and the goal of wastewater renovation and reuse.

As a result of the research activities fostered by this program, new treatment techniques have been developed and the performance of existing processes improved. In this paper a general review of the advanced waste treatment program will be given with special emphasis on those developments that are ready for full-scale engineering application.

WASTEWATER CHARACTERISTICS

An excellent starting point for this discussion is a review of the characteristics of wastewater. Contrary to popular belief, sewage is only slightly contaminated water. In fact, the usual domestic wastewater in this country is 99.95 percent water. Unfortunately, for most uses water must contain much lower levels of contamination. Thus, although waste treatment technology need only extract small quantities of contaminants from wastewater, it must reduce these contaminants to very low levels. This procedure is inherently expensive and places waste treatment technology at a disadvantage in comparison with other areas of industrial chemical processing.

A second major characteristic of wastewater which must be considered is the great diversity of specific compounds which make up the contaminants. In fact, the variety of contaminants is so great and the contaminant concentration so low that only a few substances exist at a measurable level. Much effort is being expended on developing reliable analytical methodology for rigorous analysis of wastewater; however, success is far in the future. Consequently,

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wastewater analysis is based on identifying classes of substances which have similar environmental effects. The six major classes of contaminants are: suspended solids, organics (BOD, COD, TOC), phosphorus compounds, nitrogen compounds, microorganisms, and salts. The adverse environmental effects of each class of substances will be discussed in detail later in this paper.

It will be noted that the terms BOD, COD, and TOC are in parentheses next to the general term organics. These are the name of the three most common tests utilized to measure the organic content of wastewater. BOD measures organics in terms of the oxygen required by microorganisms to oxidize the organics, COD measures the oxygen required to chemically oxidize the organics, and TOC directly measures the amount of organic carbon present. None of these tests give a complete determination of organics in wastewater. Over the last few years the TOC test has started to gain predominance over the other tests because of the rapidity with which it can be conducted.

CONVENTIONAL TREATMENT TECHNOLOGY

At present, the accepted method of wastewater treatment is that illustrated in Figure 1. This system is referred to by a variety of titles such as: conventional treatment, biological treatment and primary plus secondary treatment. This treatment train has been specifically developed to remove suspended solids, biodegradable organics and microorganisms from wastewater.

Suspended solids produce sludge banks in rivers, biodegradable organics lower the oxygen resources of lakes and rivers and microorganisms are the source of most water-borne disease. In the past, it was only necessary to substantially remove these three classes of pollutants from wastewater prior to discharge to avoid adverse environmental effects. This is no longer generally true because of expansion in population, expansion of industry, and much greater demand for recreational and esthetic use of water resources.

Although in many locations the conventional system no longer is sufficient of itself, it is the base upon which some newer, more effective treatment systems are constructed. Therefore, a short description of this system and the degree of treatment which it can provide is given below.

As illustrated in Figure 1, wastewater is first passed through preliminary treatment of screening and grit removal. Preliminary treatment is utilized to

BIOLOGICAL TREATMENT SYSTEM
-Primary & Secondary-

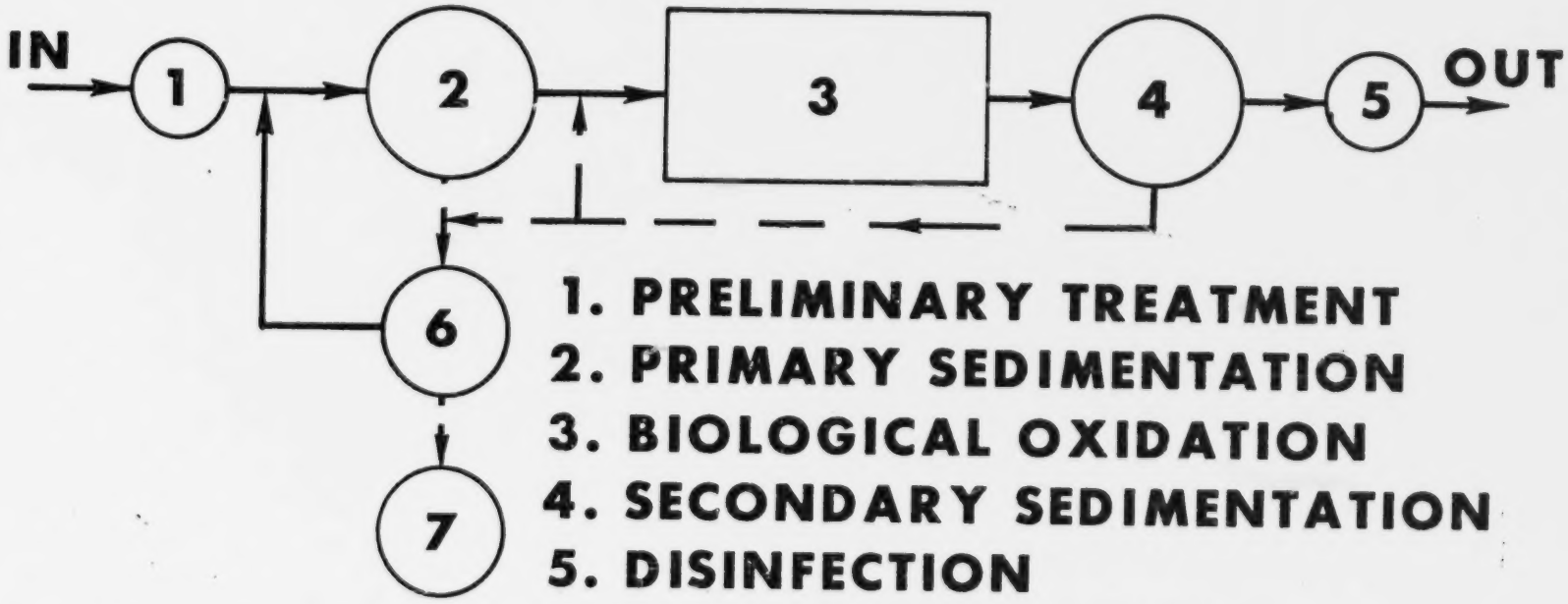


FIGURE 1

protect pumps and pipes downstream from harm from large articles and abrasives which are often found in sewage. Next, primary sedimentation is provided to remove relatively large organic solids. The following step is biological oxidation in which a large mass of microorganisms is contacted with the sewage in an aerobic environment. In this step, the microbes consume the soluble and colloidal organics producing more microbes and carbon dioxide and water. Two systems which are used for biological oxidation are the trickling filter and activated sludge. Details of these will not be discussed here. The microbes which are active in biological oxidation are separated from the flow in the secondary sedimentation tank. Being organic matter, they cannot be discharged with the effluent. Some are recycled to the head end of the biological oxidation process in order to maintain an adequate population in this unit, the remainder are sent to the sludge handling section. After passage through the secondary sedimentation tank the flow is disinfected, usually by the application of chlorine and discharged. No attempt is made to sterilize the effluent since all that is required is elimination of pathogens (disease producing microbes). Most pathogens are more susceptible to the effects of chemical disinfectants than the non-pathogens. It is important to note that the microbes which function in the biological oxidation process are non-pathogens.

When considering the cost and effectiveness of waste treatment technology sludge disposal is often overlooked. This is unfortunate as up to half the cost of the conventional treatment train can be charged to sludge handling and disposal. Sludge is a thick suspension of organic solids which is drawn from the bottom of the primary and secondary clarifiers. Disposal usually involves two steps, the first of which is dewatering. Dewatering is generally necessary because the cost of the second step "ultimate disposal" is proportional to the volume of the sludge. Unfortunately, sludge is usually difficult to dewater unless chemicals are added and/or it is subjected to anaerobic biological treatment. The liquid removed during the dewatering step is referred to as supernatant and is usually recycled to the initial portion of the treatment plant (i.e., path from 6 to 1-2 in Figure 1).

Among the methods employed for ultimate disposal are land fill, land spreading, incineration, wet oxidation, and processing for recovery of useful byproducts. At present, research in ultimate disposal is aimed at reduction of costs of environmentally acceptable ultimate disposal methods.

Table 1 illustrates the performance which can be expected of a typical well-operated conventional treatment plant. The results have been presented both as percent removal and typical effluent characteristics. As indicated previously, this type of plant cannot provide significant removal of phosphorus, nitrogen or salts; suspended solids, organic removal and microorganism removal are significant. In general, the effluent characteristics column is a more fundamental testament of the potential of this process scheme rather than the percent removal column.

TABLE 1

TYPICAL PERFORMANCE - CONVENTIONAL TREATMENT

Pollutant	Effluent mg/l	% Removal
Suspended Solids	20-30	80-90
BOD	15-25	80-90
COD	30-60	70-80
Ammonia-N	15-25	0-10
Phosphorus	6-10	0-40
Coliform	1 per ml	99.999

The cost of conventional treatment expressed as cents per thousand gallons is presented in Table 2. These costs represent the total waste treatment cost, i. e., capital amortization, operation and maintenance. All cost data in this paper were prepared by the Treatment Optimization Group of the Advanced Waste Treatment Program. A uniform amortization schedule was used and costs are referenced to October 1970. The data in Table 2 include costs for a wide size range of plants with land disposal or incineration for sludge

TABLE 2

TREATMENT COST CONVENTIONAL SYSTEM

Total Plant Cost - Cents/1000 Gal. as a Function of Sludge Disposal Procedure

Plant Size, MGD	Land Disposal	Incineration
1	36	56
10	11.5	17
100	6.5	8

disposal. Incineration is the most expensive form of ultimate disposal, and land disposal is the least costly,thus these costs bracket all usual situations.

BIOLOGICAL-PHYSICAL TREATMENT

The inability of gravity sedimentation in the final clarifiers to remove small, light particles places a limit on the capability of conventional treatment to reduce suspended solids and BOD in the treatment of wastewater. The usual range of this limit is indicated by the figures presented for typical effluent characteristics in Table 1. Integration of a better liquid-solid separation device into the treatment scheme is thus one method of up-grading conventional treatment. Figure 2 illustrates the simplest type of advanced waste treatment, the biological-physical treatment system. This system is identical to the conventional treatment system with the exception of the provision of an additional solids removal step after secondary sedimentation. Three types of systems have been widely used to upgrade the performance of conventional systems. Each will be discussed below.

Microstrainers

The simplest system is the microstrainer, illustrated in Figure 3. Microstrainers are rotating drums on which woven filter fabrics, usually of stainless steel, are mounted. Incoming wastewater flows into the drums along the axis and then passes through the filter cloth which forms the drum surface.

BIOLOGICAL-PHYSICAL TREATMENT SYSTEM

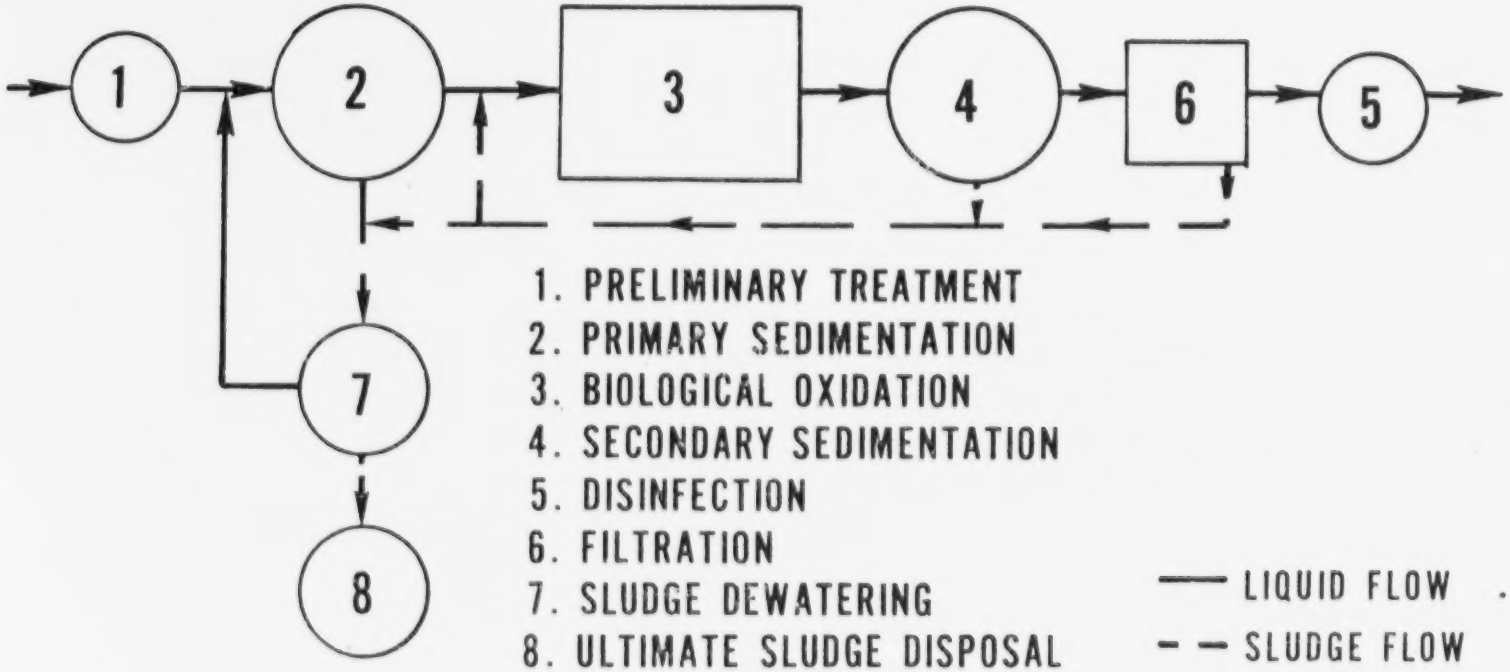
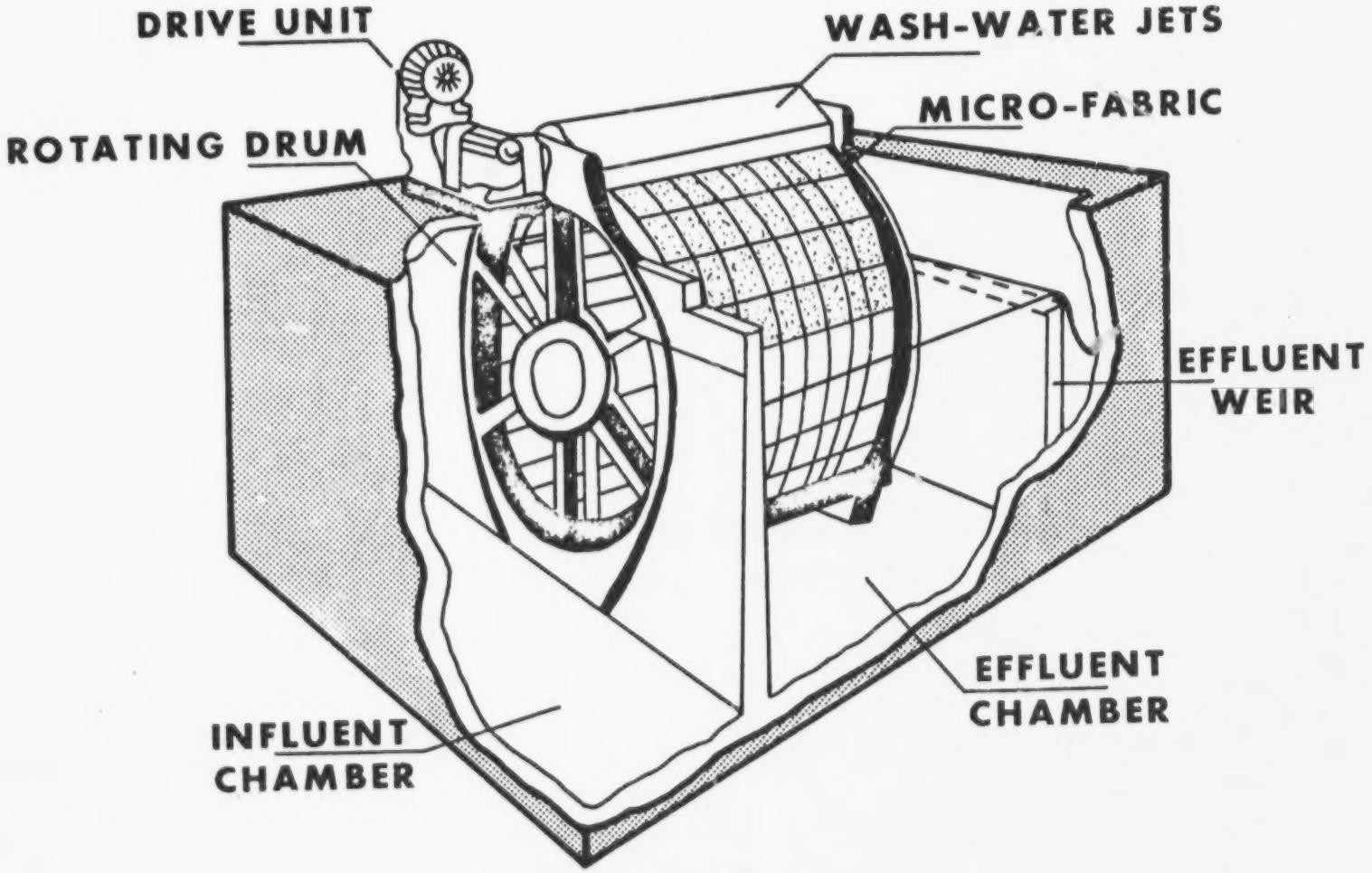


FIGURE 2



TYPICAL MICROSTRAINER UNIT

FIGURE 3

Filter rates are 6-10 gpm/ft^2 with head differential at about 6 inches of water. Control of headloss is obtained by washing the solids from the screen as that portion of the screen rotates to the top of the device. The backflow liquid discharges to a trough in the interior of the drum from which it is recycled to the sedimentation basin. Ultraviolet light mounted above the screen is used to prevent biological growth from blinding the screen.

Microstraining is fundamentally a screening process, thus the performance is a function of screen size. Table 3 illustrates this effect[1]. It can be seen that substantial improvement in effluent characteristics can be obtained by this technique. BOD removal closely parallels suspended solids removal as most of the BOD exerted by a conventional plant effluent is due to biological floc which escaped removal in the secondary tank. An extensive study of microstrainers was conducted in Chicago[2]; based on this study a 15 MGD plant was designed and is now being installed.

Cost data which will be presented later indicate that a microstrainer installation is the most economical of the upgrading techniques. However, microstrainers do not perform well under shock load conditions.

TABLE 3

MICROSTRAINER PERFORMANCE ON SECONDARY EFFLUENT

Location	Screen Size Microns	Suspended Solids % Removal	BOD % Removal
Luton, England	35	55	30
Bracknell, England	35	66	32
Chicago, Illinois	23	71	74
Brampton, Ontario	23	57	54

Deep Bed Filtration

A somewhat more costly but more reliable upgrading technique is the use of deep bed filtration. A typical deep-bed filter installation is shown in Figure 4. In this technique, the wastewater is forced at 2 to 10 gpm/ft^2 through several feet of a granular material wherein the suspended solids are deposited. Most of the removal takes place on the surfaces of the grains rather than in the bed pores. The accumulation of solids in the bed eventually produces an excessive pressure drop. At this point, the filtration run is terminated, and the bed is cleaned. Cleaning always involves an upward flow of water at a rate sufficient to expand the media, thus allowing for removal of the entrapped floc. In addition, an air scour and/or a water jet surface wash may be employed to remove solids which adhere to the grains. The nature of wastewater floc is such that it is almost mandatory for air scour or surface wash to supplement normal water backwash in order to achieve adequate cleaning.

Filter media ranges in size from 10 mesh to 80 mesh and until recently only a single type of media has been employed. After backwash, however, a single medium filter is graded with the smallest size particles at the surface. This restricts filtration to the top layers of the bed. To counteract this effect dual-media or multi-media filter beds are now coming into use. These employ granular materials of such sizes and specific gravities that after backwash the coarser material is always at the top of the filter. Typical combinations are coarse coal and fine sand or coarse coal, medium sand and fine garnet. These new combinations allow much longer filter runs at higher flow rates than single medium systems.

Typical performance of deep-bed filter systems for treatment of secondary effluent is given in Table 4[3]. Removals of pollutants are only slightly better than with microstrainers, but deep-bed filters react well to shock loads. Cost data for deep-bed filter systems will be discussed in a later section of this paper.

Chemical Treatment

Suspended solids and BOD removal across a microstrainer or deep-bed filter will rarely exceed 80 percent because these processes cannot remove colloidal matter from wastewater. Chemical coagulation is the only feasible method for

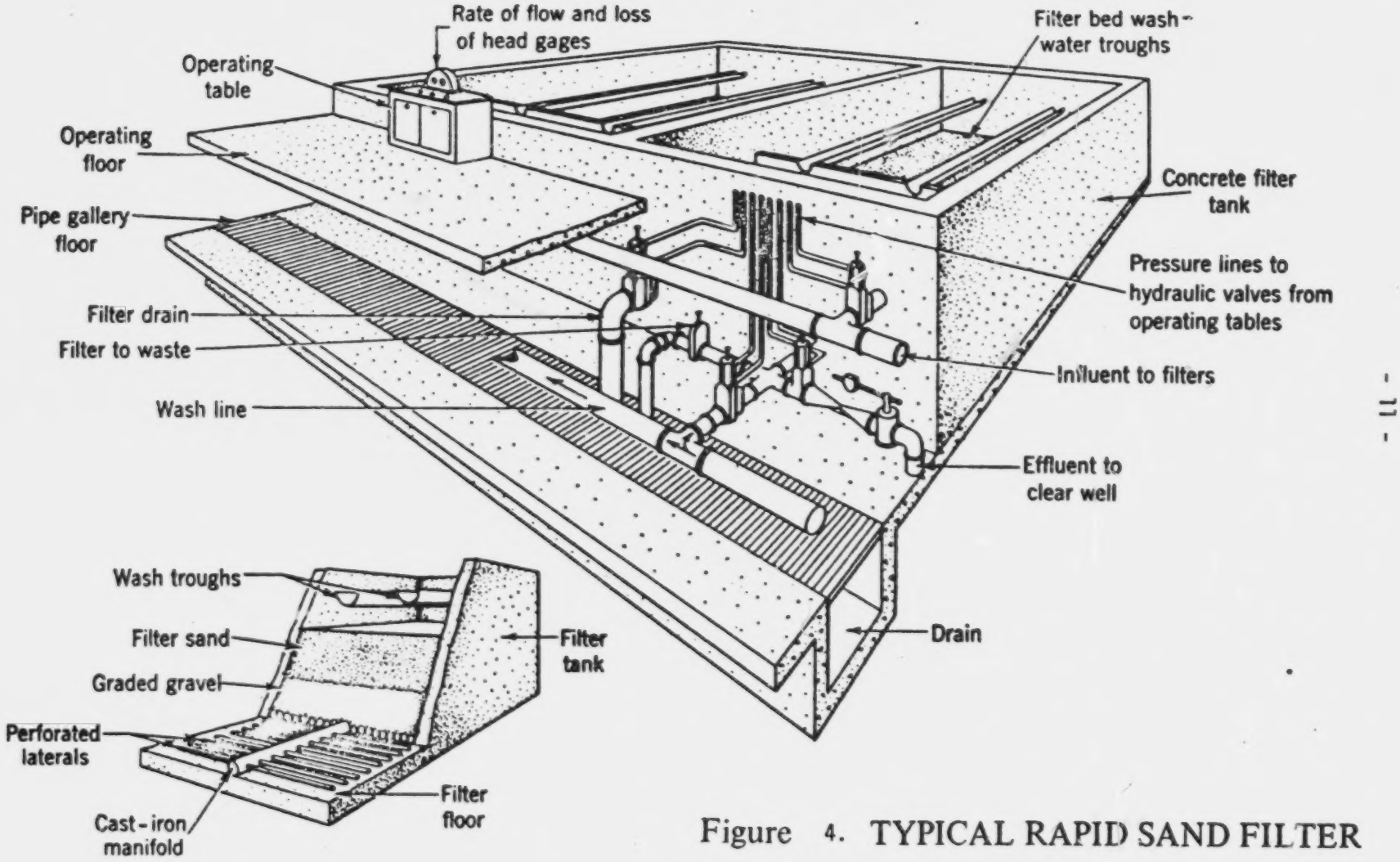


Figure 4. TYPICAL RAPID SAND FILTER

TABLE 4

DEEP BED FILTRATION - SECONDARY EFFLUENT

	Flow Rate gal/min/ft^2	Suspended Solids % Removal	BOD % Removal
Luton, England	1.5-4	72-91	52-70
Chicago, Illinois	2-6	70	80
Los Angeles, Calif.	-	46	57
Tudor, England	-	85	-

removal of colloids. In this process, chemicals added to the wastewater entrap the colloids into a floc which can then be removed by sedimentation or filtration. A large variety of chemical agents have been found useful as coagulants, these include salts of iron, salts of aluminum, lime and organic polymers. In addition to removing colloids, chemical treatment will coagulate the suspended solids in sewage forming large, dense flocs which are amenable to removal by sedimentation.

Chemical treatment itself only tends to entrap or coagulate colloids and suspended solids. Removal of these coagulated impurities requires liquid-solid separation techniques. Consequently, a system which employs chemical treatment for upgrading can be quite simple or quite complex depending on the degree of removal and reliability desired.

The simplest system would have chemical addition and flocculation between the biological oxidation unit and the secondary tank. A more comprehensive system would employ the system above with a deep-bed filter following secondary sedimentation. The most complex system, which is illustrated in Figure 5, provides chemical addition, flocculation, sedimentation and deep-bed filtration after the secondary sedimentation step. It will be noted that a microstrainer is not included in any of these flow schemes because it does not perform well with a chemical floc.

Table 5 summarizes data from a number of installations in which chemical treatment is practiced as an upgrading technique.

TYPICAL FLOW DIAGRAM OF CLARIFICATION SYSTEM

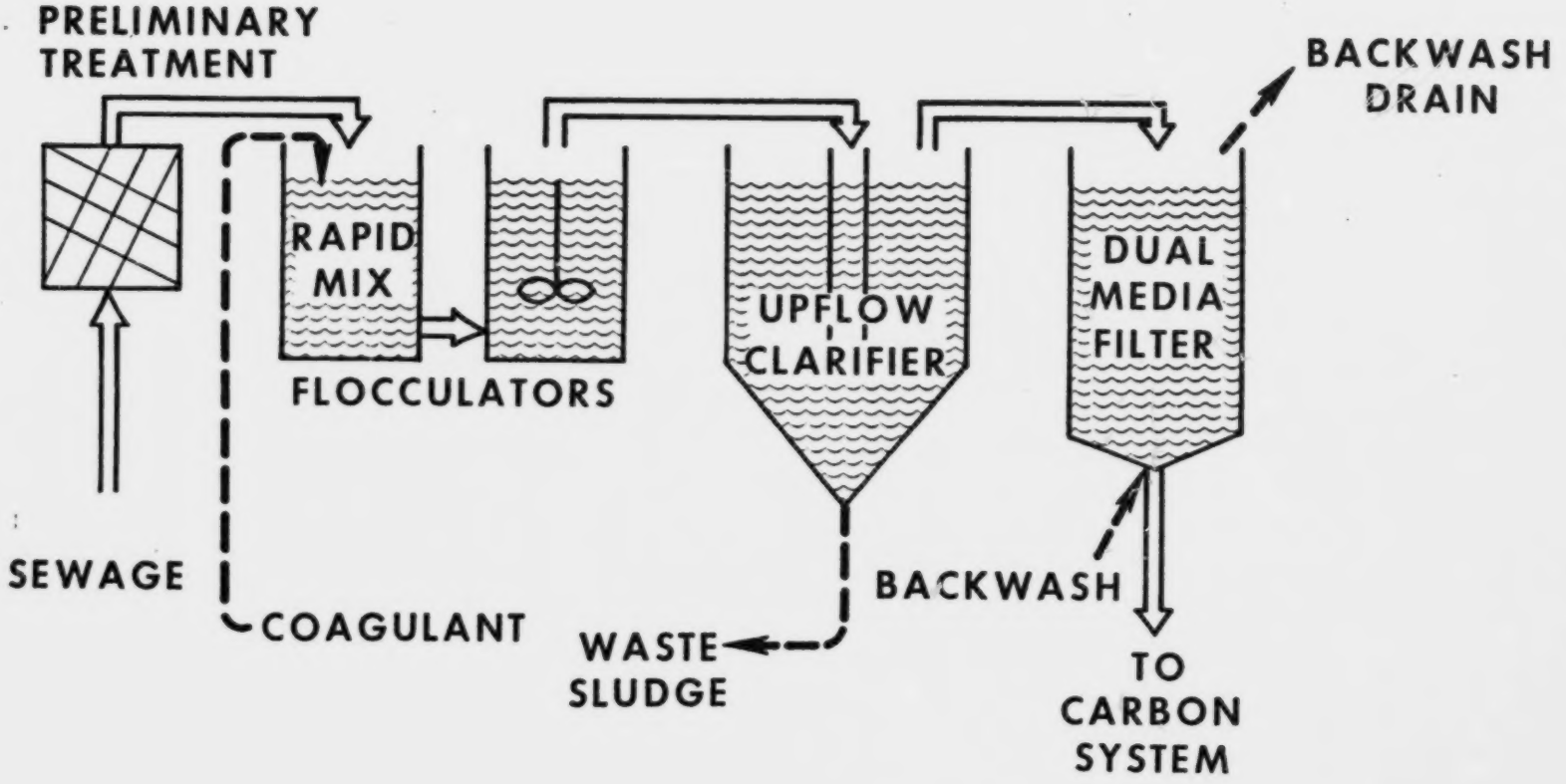


FIGURE 5

TABLE 5

CHEMICAL TREATMENT OF SECONDARY EFFLUENT

Location	Treatment*	Effluent Quality	Reference
Nassau County, N.Y.	C, (F), S, F	0.4 JTU	(4)
Lake Tahoe, Calif.	C, (F), S, F	0.3 JTU, 5 BOD	(5)
Lebanon, Ohio	C, (F), S. F	0.07 to 0.14 JTU	(6)
Washington, D.C.	C, (F), S. F	6 mg/l S.S., 6 BOD	(7)
Richardson, Texas	C, S	7 mg/l S.S., 5.5 BOD	(8)

*C = coagulation; S = sedimentation; (F)flocculation; F = filtration

TABLE 6

TREATMENT COST FOR UPGRADING CONVENTIONAL SYSTEMS

BY SUSPENDED SOLIDS REMOVAL

Process	Cost in cents per 1000 gallons Plant Size, MGD		
	1	10	100
Microstrainer	1.4	1.1	0.9
Deep Bed Filter	6.8	3.0	1.4
Chemical Coagulation-Flocculation	6-20	3-10	2-5

Estimated capital, operation and maintenance costs for the three types of biological-physical and upgrading techniques are given in Table 6. Microstraining, which is the least versatile system, is the cheapest and the cost is almost independent of plant size. Although chemical treatment systems are the most versatile and reliable, and give the highest degree of treatment, they are the most expensive. A range of costs is presented for the chemical treatment systems at each plant size because of the variety of flow schemes included

in this class. The low cost represents the system when chemical is applied after biological treatment and all liquid-solid separation takes place in the secondary tank. The highest cost presented is for the system which employs chemical addition, flocculation, sedimentation and filtration after secondary sedimentation. Although the chemical treatment system can be much more expensive than microstrainers or deep-bed filters, it must be the system of choice where a high degree of colloidal material exists in the waste water. This is often the case when the biological system is overloaded.

PHOSPHORUS REMOVAL

The key role of phosphorus in the process of eutrophication (aging of lakes and impoundments) has been known for many years. However, until recently, eutrophication of the nation's waterways was not a significant problem. Consequently, control of the phosphorus level in streams and lakes was not considered an important pollution control problem. Since the conclusion of World War II, however, the rate of eutrophication has increased to the point where it is a major water quality problem. Although other nutrients play a role in eutrophication, recent articles by Sawyer[9,10] have indicated that much of the recent increase in eutrophication rate is linked to significant increases in phosphorus discharges to rivers and lakes. Sawyer has shown that virtually all of the increase in phosphorus discharge is due to the activities of man and has termed the resulting eutrophication, cultural eutrophication.

The major sources of phosphorus contributing to eutrophication are domestic sewage and agricultural runoff. Domestic sewage is the primary source in critical areas, and only this source will be discussed here. Phosphorus gains entrance to sewage from human body wastes (primarily urine) and through the use of condensed inorganic phosphate compounds as builders in detergent formulations. Each of these sources accounts for about half of the phosphorus in domestic sewage. Thus, while elimination of phosphorus from detergent formulations would be helpful, it would not be the total answer to the eutrophication problem. Treatment of domestic sewage to remove a significant portion of the phosphorus contributed by human wastes and detergent builders would, however, have a significant effect on eutrophication rate.

Phosphorus Removal in Conventional Treatment

Removal of any pollutant from wastewater requires that it be converted to either an insoluble gas or an insoluble solid. Because none of the chemically stable forms of phosphorus is a gas at normal temperature and pressure, removal from wastewater is dependent on formation of an insoluble solid. Less than 10 percent of the phosphorus discharged to municipal sewerage systems is insoluble and none of the conventional treatment techniques is particularly effective in insolubilizing this nutrient. Thus, phosphorus removal in conventional treatment systems is relatively poor. Primary treatment can remove only the 10 percent of the phosphorus which is initially insoluble. During secondary treatment, phosphorus removal is achieved by synthesis into the biomass followed by sedimentation and sludge wasting. However, municipal sewage contains a considerable excess of phosphorus over that required for biomass synthesis during complete utilization of the organic carbon present; thus, removals are generally limited to 20-40 percent[11]. Studies indicate that biological systems have the capacity for much higher removals through the mechanism of "luxury uptake"[12]. However, attempts to implement this phenomenon in actual plants have not been successful[13]. Indeed, a recent study has illustrated that luxury uptake may actually be a chemical precipitation rather than a biologically mediated phenomenon[14].

Phosphorus Removal by Chemical Precipitation

Fortunately, phosphorus forms essentially completely insoluble precipitates with a number of substances, thus high levels of removal can be obtained when appropriate doses of the proper chemicals are applied. A large variety of chemicals can be utilized for this purpose but economic factors dictate the use of salts of iron, salts of aluminum or lime.

For both the salts of aluminum and iron, the required dose is set by similar factors. The major control is the stoichiometric reaction between the metal salt and the phosphorus compound as illustrated in equation 1.

$$M^{+3} + PO_4^{-3} \text{ ----------> } MPO_4 \downarrow \qquad (1)$$

This reaction indicates a requirement of 1 mole of the metallic cation for each mole of phosphorus in the wastewater. However, a number of factors act to

increase the dose of metallic salt required. As indicated by equation 2, the metal salt reacts with the alkalinity in the wastewater to form insoluble hydroxide.

$$M^{+3} + 3HCO_3^{-1} \text{ --------> } M(OH)_3\downarrow + 3CO_2 \qquad (2)$$

Although this is a competing reaction with the phosphorus precipitation, the metal consumed is not wasted as the hydroxide formed is an excellent flocculant of the phosphorus precipitates of equation 1. Another factor which tends to increase metal dosage over that theoretically required is pH. Figure 6 illustrates that aluminum phosphate is least soluble at pH 6 and iron phosphate is best precipitated at pH 4[15]. It is usually best to use additional metal salt rather than adjust the pH of the sewage from its normal range of 7-8. Another factor which influences coagulant dose is the presence of suspended organic solids in the wastewater. A portion of the metal salt added will be required for coagulation of these materials, again increasing the dose requirement.

The dose required to achieve any specified degree of phosphorus removal cannot be specified with great precision because of the factors discussed above. It is best to conduct bench-scale jar tests on the wastewater to be treated to establish the metal salt dose requirement. Average of results from many installations which can serve as a rough guide are given in Table 7[11].

TABLE 7

AVERAGE METAL SALT DOSE REQUIRED FOR
SPECIFIED PERCENT PHOSPHORUS REMOVAL

% P Removal	Dose Metal Salt Mole Ratio M^{+3}/P
75	1.4
85	1.7
95	2.3

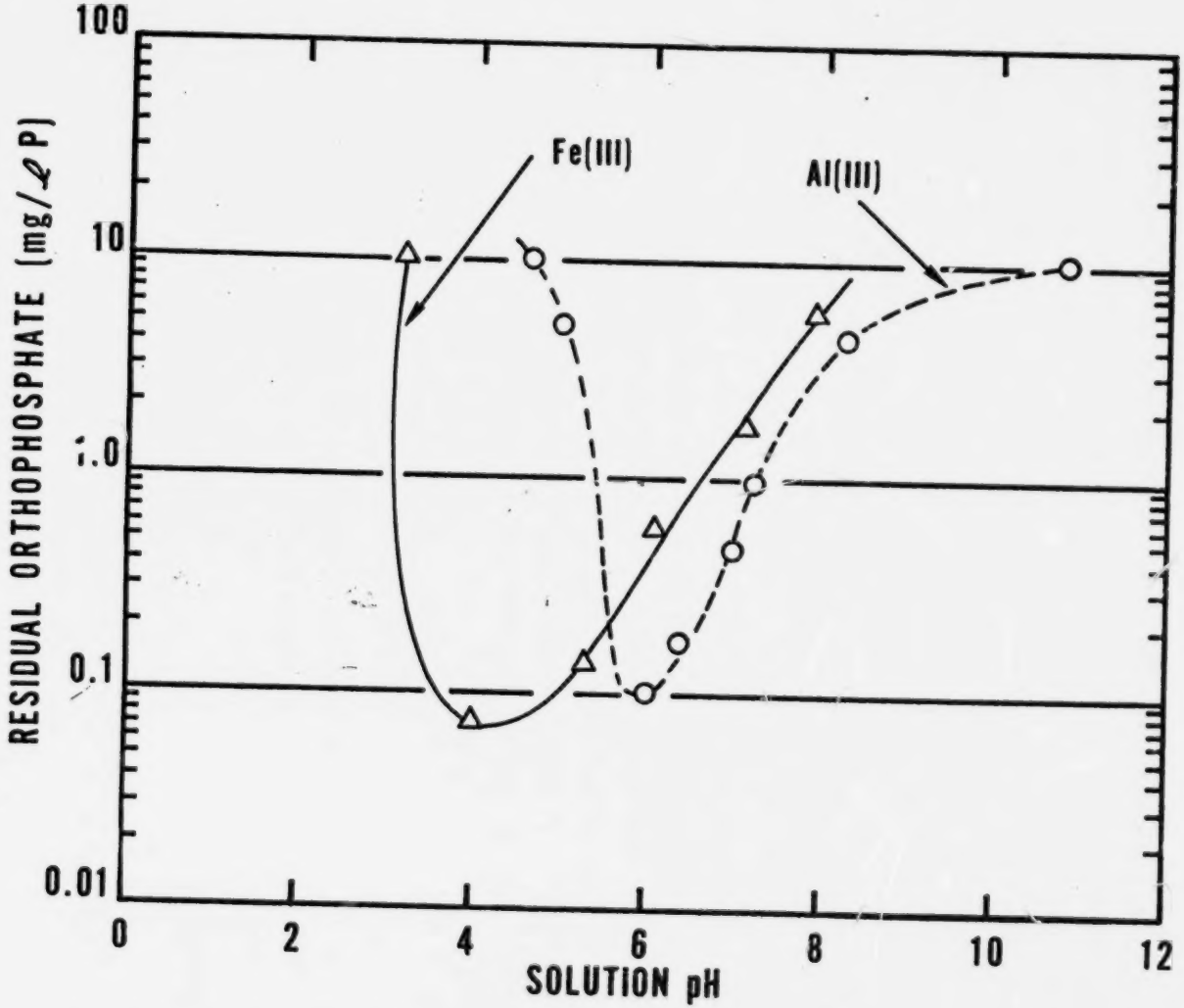


FIGURE 6

PRECIPITATION OF ORTHOPHOSPHATE WITH Al (III) AND Fe (III) AT A 2:1 CATION-TO-ORTHOPHOSPHATE MOLAR RATIO (Ref. 17)

The reaction of lime with wastewater constituents is given in equations 3 and 4:

$$3HPO_4^{-2} + 5Ca^{+2} + 4OH^{-1} \text{ -----> } Ca_5(OH)(PO_4)_3\downarrow + 3H_2O \quad (3)$$

$$Ca(OH)_2 + Ca(HCO_3)_2 \text{ ---------> } 2CaCO_3\downarrow + 2H_2O \quad (4)$$

Equation 3 represents the formation of the insoluble compound calcium hydroxyapatite. The kinetics of this reaction as well as the stoichiometry are pH dependent. Elevated pH values 9.5-11.5 are required to insure good phosphorus removal by precipitation and liquid solids separation techniques. Figure 7 illustrates this pH effect[16]. Thus, while pH was a secondary factor in establishing the dose requirement with iron or aluminum as the precipitant, it is the primary factor when calcium is utilized.

In order to reach the pH range which will insure good phosphorus removals, the alkalinity demand of the wastewater must be met, as illustrated in equation 4. Because the bicarbonate concentration in wastewater is much higher than the phosphorus concentration (several hundred mg/l vs. $\approx$ 10 mg/l), therefore, most of the lime dose required to achieve effective phosphorus removal is used to meet the alkalinity demand. Here a situation exists similar to that occurring when metal salts are used for phosphorus removal; i.e., the product of the cation alkalinity reaction serves as a flocculant of the phosphorus precipitate. Thus, the reaction of the lime with the alkalinity does not completely waste the lime.

When reaction 4 is complete, the pH of the wastewater is in the range of 9.5-10.0. For wastewater with moderate to high levels of alkalinity (> 200 mg/l), sufficient calcium carbonate is formed under these conditions to effectively bring down the hydroxyapatite. After sedimentation, the pH is reduced to < 8 by the addition of CO_2. This process is referred to as single-stage lime precipitation. If the alkalinity of the wastewater is low > 150 mg/l, however, insufficient calcium carbonate is formed. For this situation, the pH must be raised above 11 which will insure precipitation of magnesium hydroxide according to equation 5:

$$Ca(OH)_2 + Mg^{+2} \text{ --------> } Ca^{+2} + Mg(OH)_2\downarrow \quad (5)$$

LIME PRECIPITATION AND PHOSPHORUS REMOVAL FROM RAW WASTEWATER.

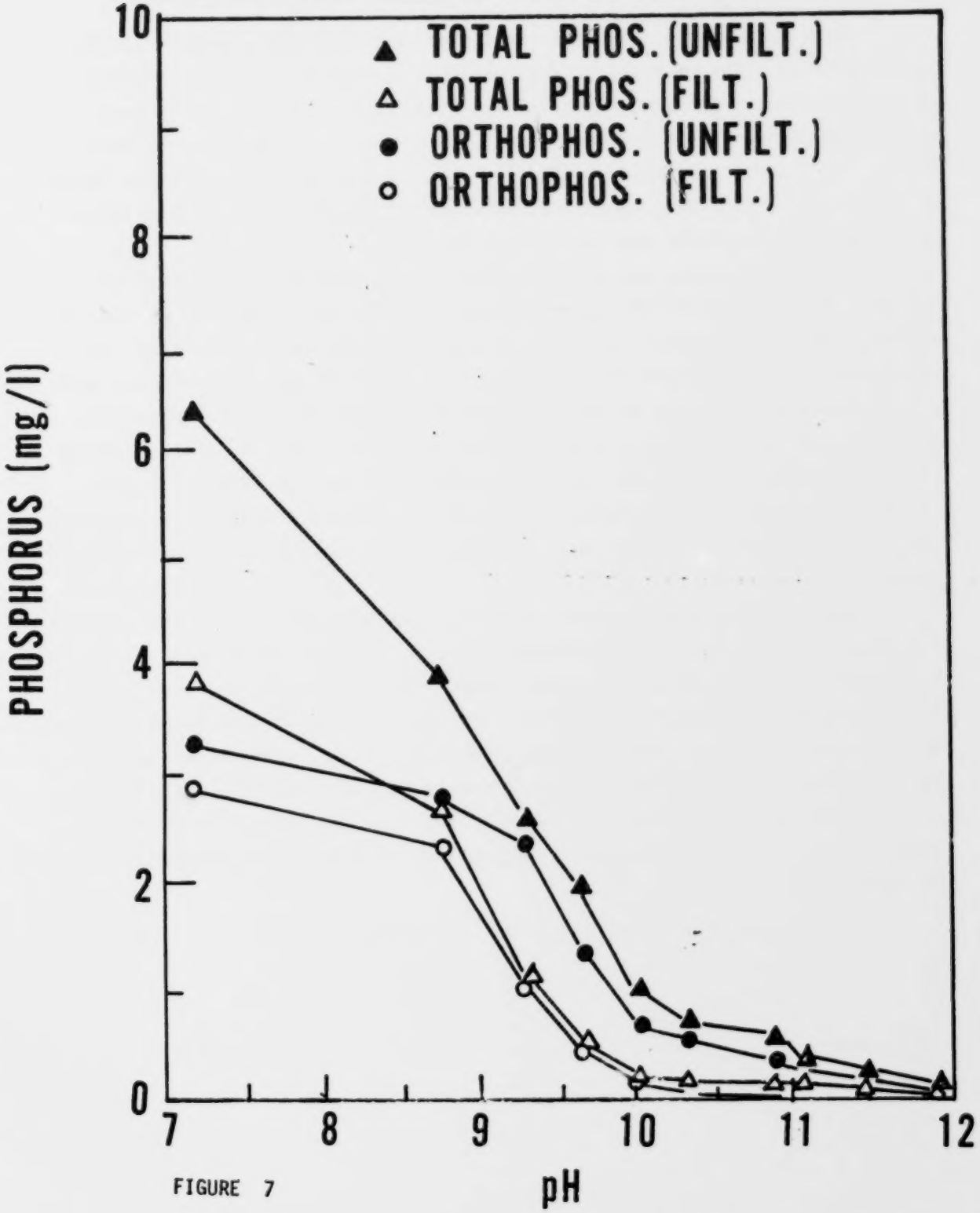


FIGURE 7

Magnesium hydroxide is a gelatinous precipitate which will effectively scavenge hydroxyapatite from solution. Adding excess lime to raise the pH to high levels results in a wastewater containing excess calcium. The calcium is reduced by carbonation with CO_2. As illustrated in equation 6, this reduces the pH and results in a second precipitation of calcium carbonate:

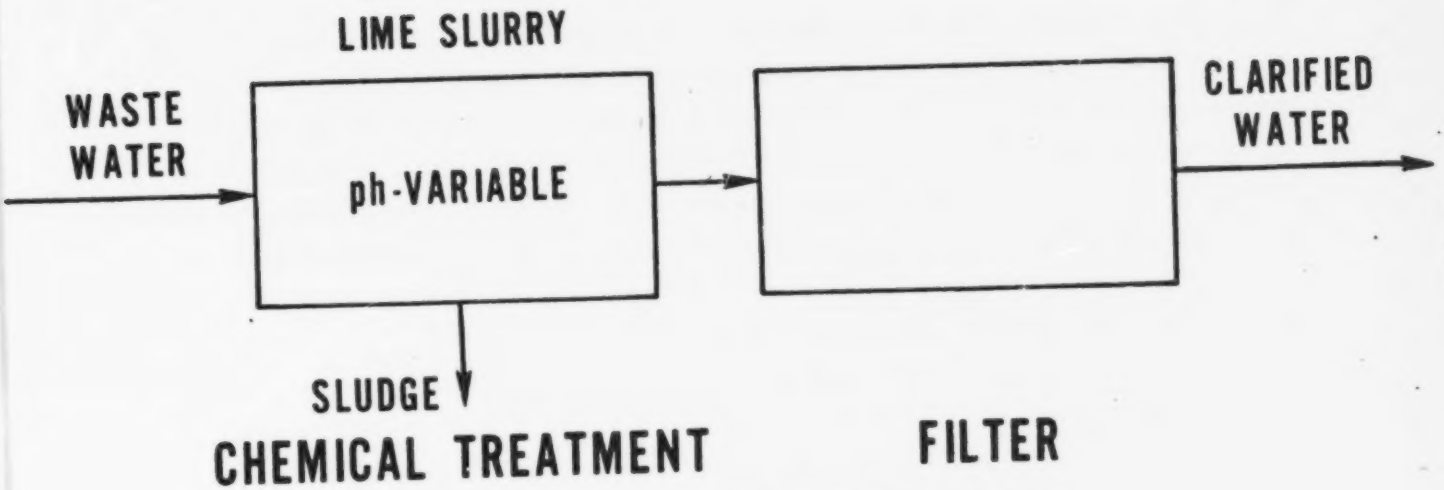
$$CO_2 + Ca^{+2} + 2\ OH^- \text{ --------> } CaCO_3 \downarrow + H_2O \qquad (6)$$

The second precipitation is carried out in a sedimentation tank separate from that in which hydroxyapatite, calcium carbonate and magnesium hydroxide precipitation takes place. The recarbonation reaction is complete at pH 9.5-10.0, which is the same end point pH as in the single stage process. A second dose of CO_2 is then applied to reduce the pH below 8. The final pH reduction in the two-stage and single-stage processes is utilized to prevent post precipitation of $CaCO_3$ in other treatment units. Flow diagrams of the single-stage and two-stage lime precipitation processes are given in Figure 8.

Table 8 gives a list of chemicals which are useful for phosphorus precipitation. Small doses of organic polymer may also be required to aid flocculation. In selecting the chemical for use at any particular site, the factors listed in Table 9 should be taken into account. It is clear that the cost of the chemical is only one factor among many which bear on the ultimate cost of phosphorus removal. Of prime importance is dewatering and ultimate disposal of sludges. Here, lime has an advantage because lime sludges are more easily dewatered than iron or aluminum sludges. In addition, lime sludge can be recalcined to reusable lime in an incinerator. However, large capital expenditures such as an incinerator can only be justified at large treatment plants.

Phosphorus removal is achieved by precipitation followed by liquid solids separation. For the most part, the usual liquid-solids separation equipment in a treatment plant can be utilized for phosphorus removal. This results in a considerable savings in capital as well as integration of phosphorus removal into conventional treatment plant operation. In addition, it has been found that the use of chemical precipitants in conventional treatment can markedly upgrade performance of the treatment plant. This results from coagulation of organic suspended and colloidal solids by the chemicals added to precipitate phosphorus.

SINGLE STAGE LIME TREATMENT



TWO STAGE LIME TREATMENT

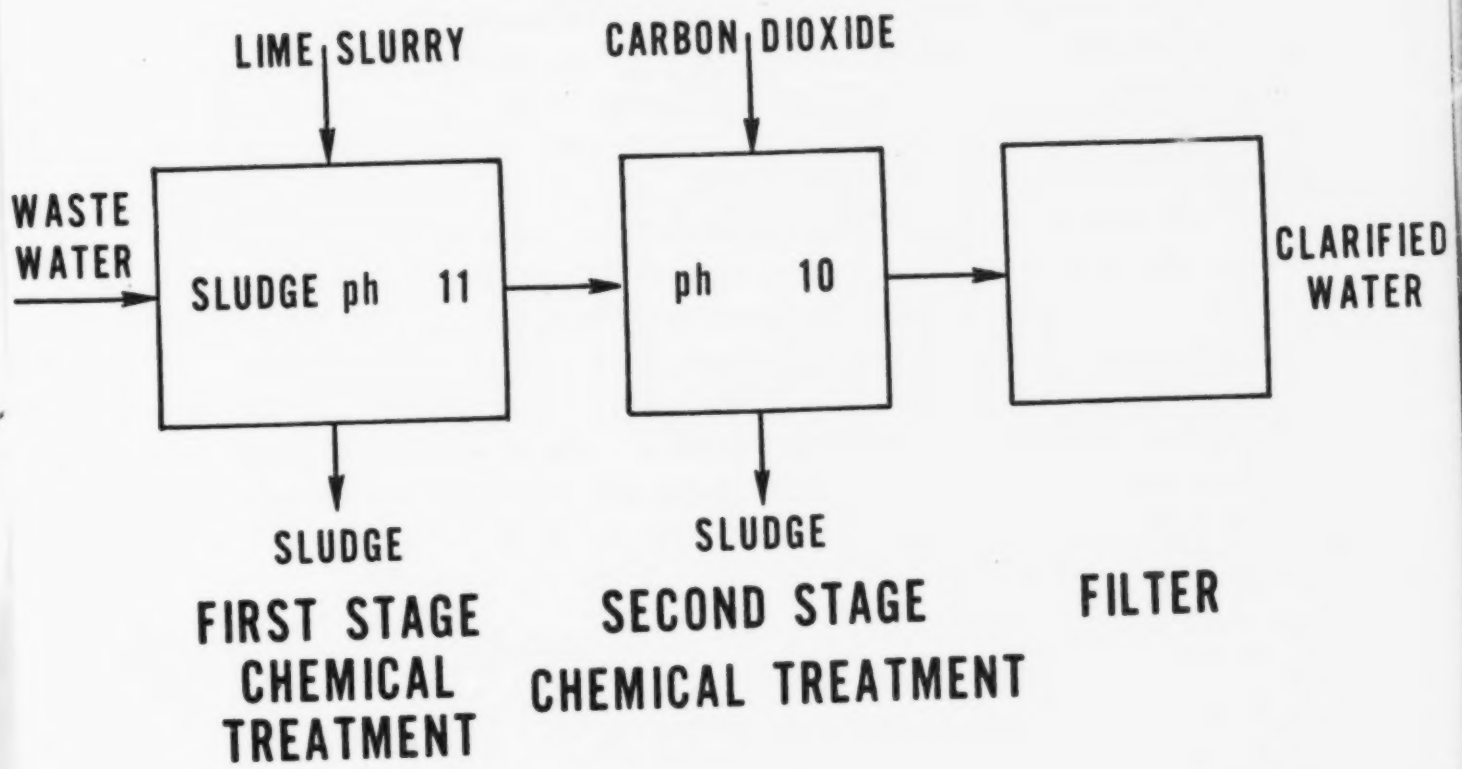


FIGURE 8

TABLE 8

CHEMICALS FOR PHOSPHORUS REMOVAL

Ferric Chloride	$FeCl_3$
Ferric Sulfate	$Fe_2(SO_4)_3$
Ferrous Chloride	$FeCl_2$
Ferrous Sulfate	$FeSO_4$
Alum	$Al_2(SO_4)_3$
Sodium Aluminate	$NaAl_2O_3$
Steel Mill Pickling Liquor	$FeCl_2 + FeSO_4$
Lime	$Ca(OH)_2$

TABLE 9

FACTORS AFFECTING CHOICE OF CHEMICAL
FOR PHOSPHORUS REMOVAL

Influent Phosphorus Level

Wastewater Suspended Solids and Alkalinity

Chemical Cost Including Transportation

Reliability of Chemical Supply

Sludge Handling Facilities

Ultimate Disposal Methods

Compatibility with Other Treatment Processes in Plant

Potential Adverse Environmental Effects

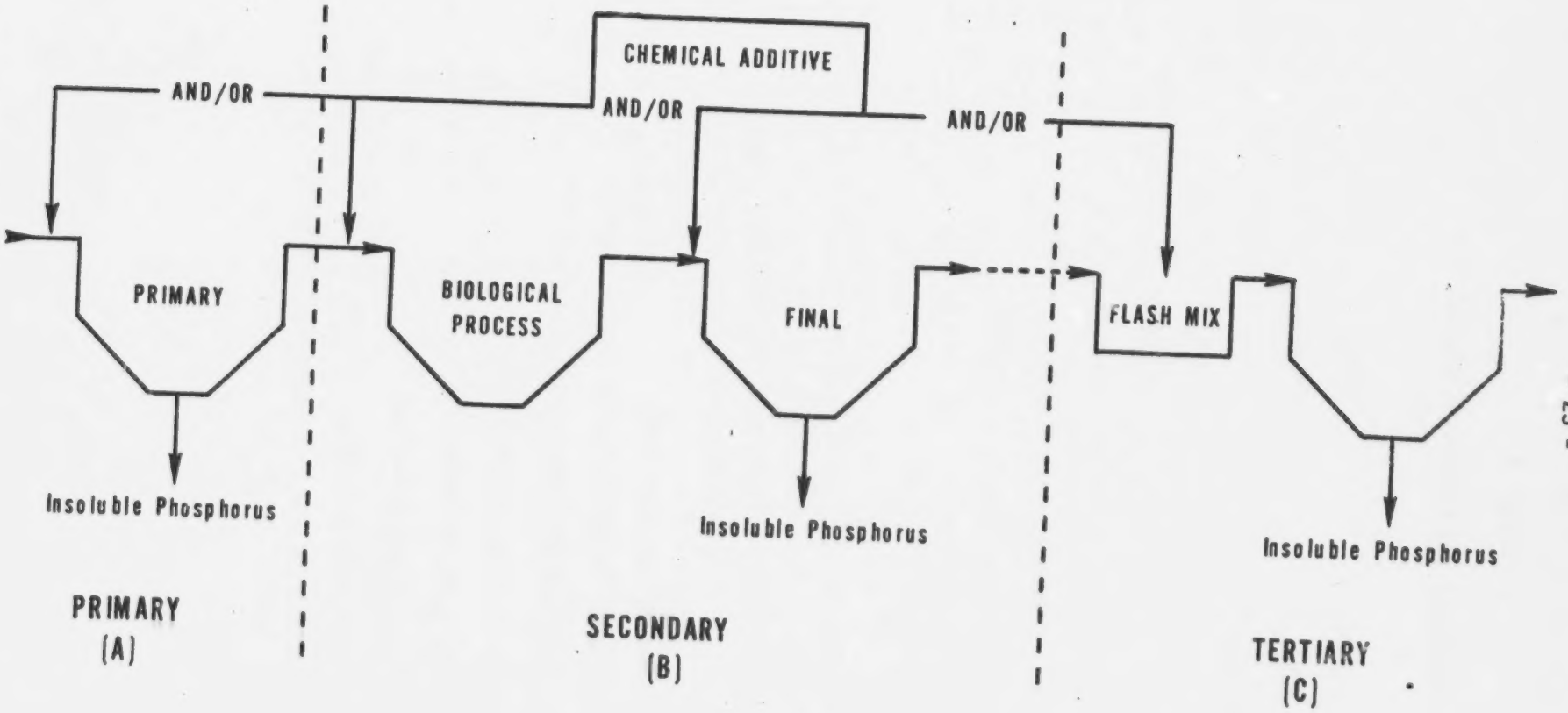


FIGURE 9

LOCATIONS FOR CHEMICAL CONTROL OF PHOSPHORUS

Figure 9 illustrates a conventional treatment plant with the three general sections in which phosphorus removal can be carried out. Chemical can be added either:

a. Just before the primary tank with removal taking place in the primary.
b. In the secondary (biological) section of the plant with removal in the secondary sedimentation tank.
c. In a tertiary stage as was discussed in the section on suspended solids removal.

Table 10 illustrates typical results obtained with phosphorus removal in the primary, secondary, or tertiary. As can be seen, good removals are obtained in all sections, however, the lowest levels of phosphorus remaining are achieved in the tertiary addition. One reason for this is that a filter is usually included in the tertiary plant, thus better removal of fine precipitates is achieved. In addition, when the flow reaches this section of the plant all the complex phosphorus forms which are more difficult to precipitate have been hydrolyzed to orthophosphate which is the easiest to precipitate.

In plants where removal in the primary was practiced a major effect to note is the significant increase in BOD and suspended solids removal achieved over the removal usually obtained in the primary tank. This may be important in helping to meet water quality standards for BOD and suspended solids if the treatment plant is overloaded.

In plants where the chemical is added in the secondary section, it has been observed that much more stable operation of the activated sludge is obtained than before chemical addition. The effect of the chemical is to weigh the sludge down, preventing its loss when a filamentous or dispersed growth predominates. Even in plants which have historically exhibited excellent performance, chemical addition has improved performance by helping maintain a higher concentration of activated sludge in the aeration tank. This is illustrated by the data in Table 11 from parallel operation for one year at Penn State University[23]. It has been found best to add the chemical between the biological reactor and the final sedimentation tank rather than at the head end of the secondary tank.

Table 12 summarizes the advantages and disadvantages of carrying out phosphorus removal in the various sections of a treatment plant.

TABLE 10

PHOSPHORUS REMOVAL EXPERIENCES

Place	Chemical	P		BOD	S.S	Reference
		% Removal	Effluent	% Removal	% Removal	
PRIMARY						
Grayling, Michigan	$FeCl_2$	72	4.4	78	58	(17)
Washington, D.C.	Lime	95	0.45	82	88	(18)
Mentor, Ohio	Pickle Liquor	83.5	-	59	74	
SECONDARY						
Pomona, Calif.	Alum	80-93				(19)
Manassas, Virginia	Alum		1.0			(20)
Richardson, Texas			0.5-0.75			(21)
TERTIARY						
Nassau County, N.Y.	Alum		0.08			(5)
Lake Tahoe, Calif.	Lime		0.14			(4)
Lebanon, Ohio	Lime		0.1-0.5			(22)

TABLE 11

COMPARISON OF ACTIVATED SLUDGE PERFORMANCE
WITH AND WITHOUT ALUM ADDITION

Parameter	Influent mg/l	Effluent, mg/l	
		Alum	Normal
S.S.	110	22	26
BOD	71	9	13
COD	172	55	68
Soluble P	6.7	0.36	6.7
Total P	10.0	1.4	7.3

TABLE 12

ADVANTAGES AND DISADVANTAGES OF P REMOVAL IN VARIOUS SECTIONS OF A TREATMENT PLANT

ADVANTAGES

Primary

1. Applicable to all plants
2. Increased BOD & Suspended Solids Removal
3. Lowest Degree of Metal Leakage

Secondary

1. Lowest Capital
2. Lower Chemical Dose than Primary
3. Improved Stability of Activated Sludge
4. Polymer not Required

Tertiary

1. Lowest P in Effluent
2. Most Effecient Metal Use
3. Lime Recovery Demonstrated

DISADVANTAGES

Primary

1. Least Efficient Utilization of Metal
2. Polymer Required for Flocculation
3. Sludge More Difficult to Dewater than Primary Sludge

Secondary

1. Careful pH Control to get P < 1 mg/l
2. Overdose of Metal may cause Low pH Toxicity
3. Cannot Use Lime Because of Excessive pH

Tertiary

1. Highest Capital Cost
2. Highest Metal Leakage

The cost of adding phosphorus removal to existing treatment plants is given in Table 13. The lowest cost option is addition in the secondary because capital expenditure for chemical feed equipment and extra sludge handling equipment are least in this mode. However, removals below 1 mg/l P are difficult. The most costly option is tertiary because of the high capital cost. However, an excellent quality effluent is obtained which is very low in phosphorus, BOD and suspended solids. Note the higher tertiary cost here than for the upgrading system. This is due to higher chemical doses, and additional sludge handling difficulties. Table 14 lists a number of plants where phosphorus removal is being conducted or is in the planning stages.

NITROGEN CONTROL

Nitrogen can exist in the aquatic environment in any one of four forms: organic-N, ammonia-N, nitrite-N, and nitrate-N. In sewage it is found primarily in the first two forms. In nature, biologically mediated reactions convert organic-N to ammonia-N which in turn is biologically oxidized to nitrite-N and nitrate-N. The latter conversions are illustrated in equations 7 and 8:

$$NH_4 + 1.5\ O_2 \xrightarrow{\text{Nitrosomonas sp.}} NO_2^- + H_2O + 2\ H^+ \qquad (7)$$

$$NO_2 + 0.5\ O_2 \xrightarrow{\text{Nitrobacter sp.}} NO_3^- \qquad (8)$$

The major water quality objective of nitrogen control is to prevent excessive depletion of the dissolved oxygen resources of streams by the biological oxidation of ammonia-N to nitrate-N. Table 15 shows the significance of the nitrogen oxygen demand (NOD) of a wastewater by comparison of the oxygen demand of raw wastewater to a well-treated secondary effluent. These data illustrate that 90 percent removal of organic oxygen demand (BOD) only results in a 74 percent removal of the total biological oxygen demand (TBOD)[24]. Nitrogen control is also important because ammonia-N exerts a chlorine demand which reduces disinfection efficiency, is toxic to fish and other aquatic life and stimulates corrosion of copper plumbing.

The least expensive method of preventing these adverse actions is to carry out the biological oxidation to nitrate under controlled conditions in the treatment plant. Control of ammonia-N is possible by altering the operation

TABLE 13

ECONOMICS OF PHOSPHORUS REMOVAL

Cost Cents per 1000 Gallons

Plant Size, MGD / Chemical Addition Point	1	10	100
Primary	6-8	3-5	3-4
Secondary	6-7	3.5-4.0	3.0-3.5
Tertiary	28	12	6

TABLE 14

INSTALLATIONS PRACTICING OR PLANNING PHOSPHORUS REMOVAL

Location	Capacity, MGD	Status
Chicago, Illinois	30	Design
Seattle, Washington	20	Design
Pomona, California	2	In Operation
S. Lake Tahoe, California	7.5	In Operation
Piscattaway, Maryland	5	Construction
Colorado Springs, Colorado	2	In Operation
Richardson, Texas	1	In Operation
Blue Plains	300	In Operation
El Lago, Texas	0.5	Construction
Hatfield Twp., Pa.	5.0	In Operation

TABLE 15

OXYGEN DEMAND IN RAW AND TREATED SEWAGE

Component	Raw mg/l	Treated mg/l	% Removal
Organic Matter	250	25	90
Organic Oxygen Demand (BOD)	375	37	90
Ammonia-N	25	20	20
Nitrogen Oxygen Demand	112	90	21
Total BOD	487	127	74

of conventional activated sludge to provide for a significant degree of nitrification. The modifications involve increases in aeration rate, contact time and sludge age, and maintenance of the pH at 8.0-8.4[25]. The latter is quite important in view of the tendency of the nitrification reaction to lower the pH. These reactions are quite temperature sensitive showing significant rate decreases as the temperature drops below 18°C. As the temperature decreases, contact time and sludge age must be increased to compensate for the reduced reaction rates. Of all the factors listed above, sludge age is the most significant as it can be used to control the population of nitrifiers maintained in the system.

At low temperature and when a significant diurnal flow variation exists, it has been found difficult to maintain both organic carbon oxidation and nitrification in a single reactor. In such situations, a two-stage system employing organic carbon oxidation in the first stage followed by a nitrification stage has shown significant superiority of performance[26]. Separate sedimentation and sludge recycle is employed with each stage. This system exhibits much greater stability of performance because conditions in each stage can be established to favor the specific types of organisms desired in that stage.

Nitrification of sewage may not be sufficient for nitrogen control in some instances because an excess of nitrate in water will disqualify it from use as a potable water supply[27] and because nitrogen in any form can serve as an algal nutrient. Although phosphorus usually is the controlling nutrient in eutrophication, nitrogen controls under some circumstances. Areas where nitrogen is the controlling nutrient include San Francisco Bay, Lake Tahoe and the Potomac Estuary[28]. Removal of various forms of nitrogen from wastewater can be achieved by both biological and physical-chemical means. The available methods are discussed below.

Biological Denitrification

Under anaerobic conditions facultative heterotrophs will utilize nitrate ion as a hydrogen acceptor for the degradation of organic matter. Equation 9 illustrates this reaction with methanol as the organic carbon source.

$$5\ CH_3OH + 6\ NO_3^- \text{ ----------> } 5\ CO_2 + 3\ N_2 + 7\ H_2 + 6\ OH^- \qquad (9)$$

The end product of the nitrogen reduction is nitrogen gas which is essentially insoluble in water. Thus, a combination of nitrification followed by denitrification can achieve nitrogen removal from wastewater.

A three-stage biological system has been developed by Barth[29] to provide nitrogen removal. The first stage is a high rate short aeration time ($\approx$ 2 hours) biological reactor for organic carbon oxidation, and hydrolysis of organic nitrogen to ammonia. The second stage provides $\approx$ 3 hours of detention and achieves essentially complete nitrification. The third stage is for denitrification of nitrate to nitrogen gas. An organic source must be added to the third stage to force the denitrification reaction to take place. A variety of substances has been evaluated for this use and methanol has been found to be far superior to all others. It is relatively inexpensive, reacts rapidly, and provides only a minimum of energy for growth of new organisms. The theoretical requirement is 1.9 mg methanol per mg of nitrate-N. In practice, a dose close to 3 mg methanol to 1 mg nitrate-N is required to compensate for synthesis and the demand exerted by dissolved oxygen remaining in the wastewater after the nitrification stage.

A diagram illustrating this system is given in Figure 10. Three alternate contactors are illustrated in the denitrification stage. Contactor 1 is a suspended growth contactor similar to that used in the first two stages. This type was used in the original development by Barth. Solids control is somewhat difficult in this system, and it is often followed by a deep-bed filter. The other two systems supply a contact medium on which a heavy biological growth can develop. These latter systems require significantly less contact time than the three hours required in the suspended growth reactor, and they can accept higher hydraulic rates of application without fear of a washout of organisms. The coarse medium (1"-2" gravel) upflow system, pioneered by McCarty[30], requires a contact time of 1-2 hours and may require a supplemental filter. The fine medium downflow system developed by Smith, et al.[31], utilizes particles as small as 3 mm in beds 10-20 feet deep. Flow rate is that of a high-rate filter 7 gal/min/ft^2 with a bed contact of 10-20 min. Backwash of the fine medium units is required once or twice per day to relieve clogging. In all of the denitrification systems discussed, essentially complete denitrification has been obtained in pilot-scale experiments.

Most experience has been with the three-sludge suspended growth system. Results from a 100,000 gal/day pilot plant in Washington, D.C., are presented in Table 16. This table includes data on phosphorus removal, as addition of alum to the first stage was practiced both to maintain solids control and achieve phosphorus removal. A number of full-scale installations employing the three-sludge system with mineral addition for phosphorus removal are planned. These are given in Table 17.

BREAKPOINT CHLORINATION

Chlorine reacts with ammonia-N to yield a variety of amines and inorganic compounds by a variety of pathways. Reaction 10 below illustrate the preferred reactions for nitrogen removal:

$$3\ Cl_2 + 2\ NH_4^+ \text{ ---------> } N_2 + 8\ H^+ + 6\ Cl^- \qquad (10)$$

The reaction requires a theoretical dose of 7.6 g chlorine per gram of ammonia-N. The reaction occurs in a stepwise fashion; various chloramines initially form which in turn rapidly react with additional chlorine to yield nitrogen gas.

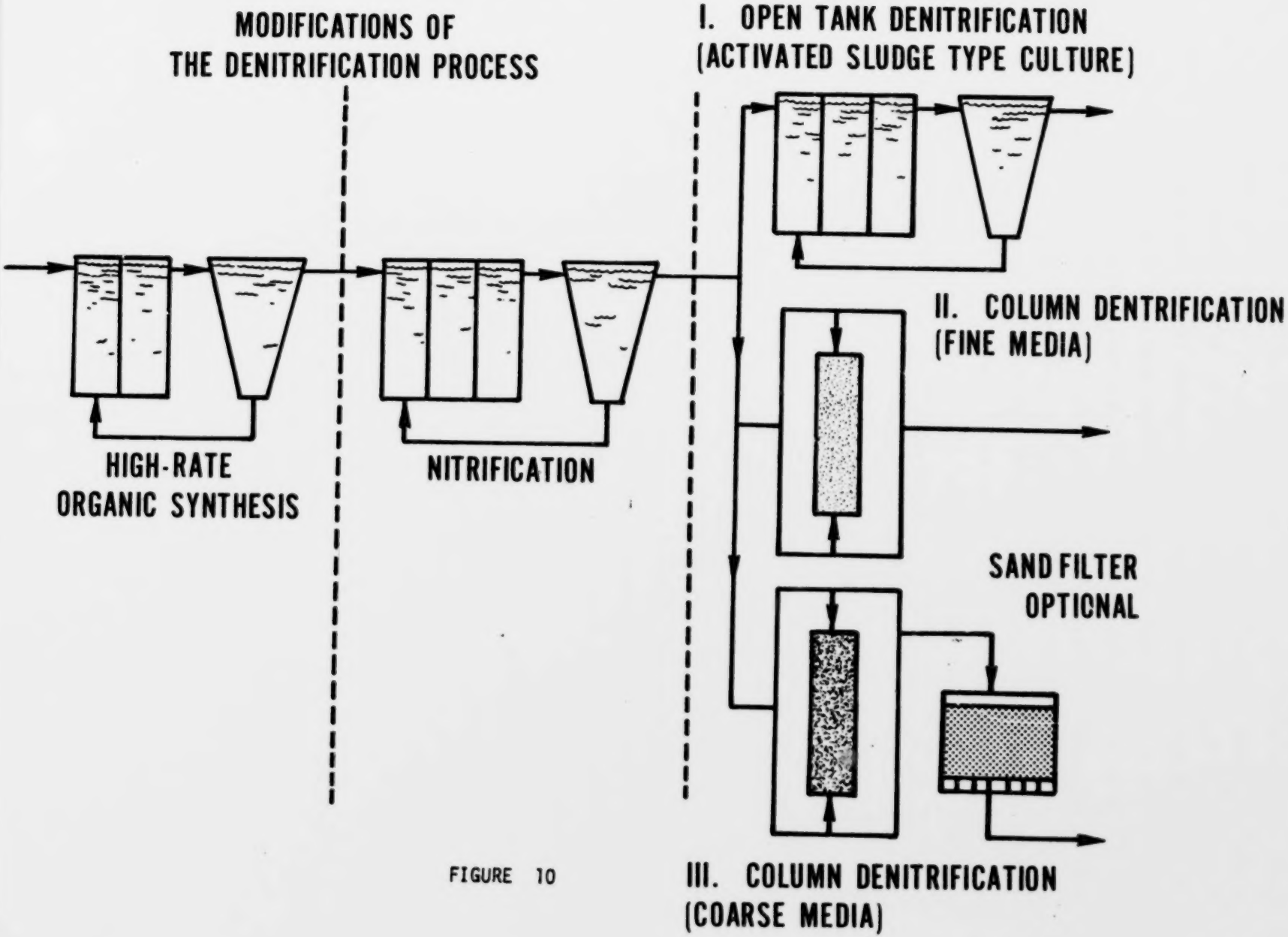

MODIFICATIONS OF
THE DENITRIFICATION PROCESS
I. OPEN TANK DENITRIFICATION
(ACTIVATED SLUDGE TYPE CULTURE)
II. COLUMN DENTRIFICATION
(FINE MEDIA)
HIGH-RATE
ORGANIC SYNTHESIS
NITRIFICATION
SAND FILTER
OPTIONAL
III. COLUMN DENITRIFICATION
(COARSE MEDIA)

FIGURE 10

TABLE 16

PERFORMANCE SUMMARY FOR THREE-STAGE BIOLOGICAL TREATMENT UTILIZING MINERAL ADDITION AND A SUSPENDED GROWTH REACTOR

Constituent	Raw Wastewater mg/l	Primary Effluent mg/l	High-Rate Effluent mg/l	Nitrified Effluent mg/l	Denitrified Effluent mg/l	Denitrified Effluent (filtered) mg/l
COD	320	218	64	43	44	38
Organic-N	10.3	5.9	0.8	0.4	0.4	0.2
Ammonia-N	11.3	13.7	7.7	0.6	0.3	0.4
Nitrite-N	-	-	1.1	0.3	0.3	0.4
Nitrate-N	-	-	4.3	11.5	0.9	0.9
Total Phosphorus	12.6	11.9	2.8	2.6	1.5	1.1
S. S.	157	90	8.8	7.1	7.1	1.7
pH (mode)	7.2	7.3	7.5	7.5	7.7	7.7

All Values are Averages Except pH and are Based on 40 Determinations

TABLE 17

PLANNED INSTALLATIONS FOR BIOLOGICAL NITRIFICATION-DENITRIFICATION SYSTEMS

LOCATION	CAPACITY, MGD
Salt Creek, Illinois	50.0
Tampa, Florida	90.0
ElLago, Texas	0.5
Clermont County, Ohio	2.0
Washington, D. C.	300.0

However, it is possible for either of the side reactions 11 and 12 to occur rather than reaction 10:

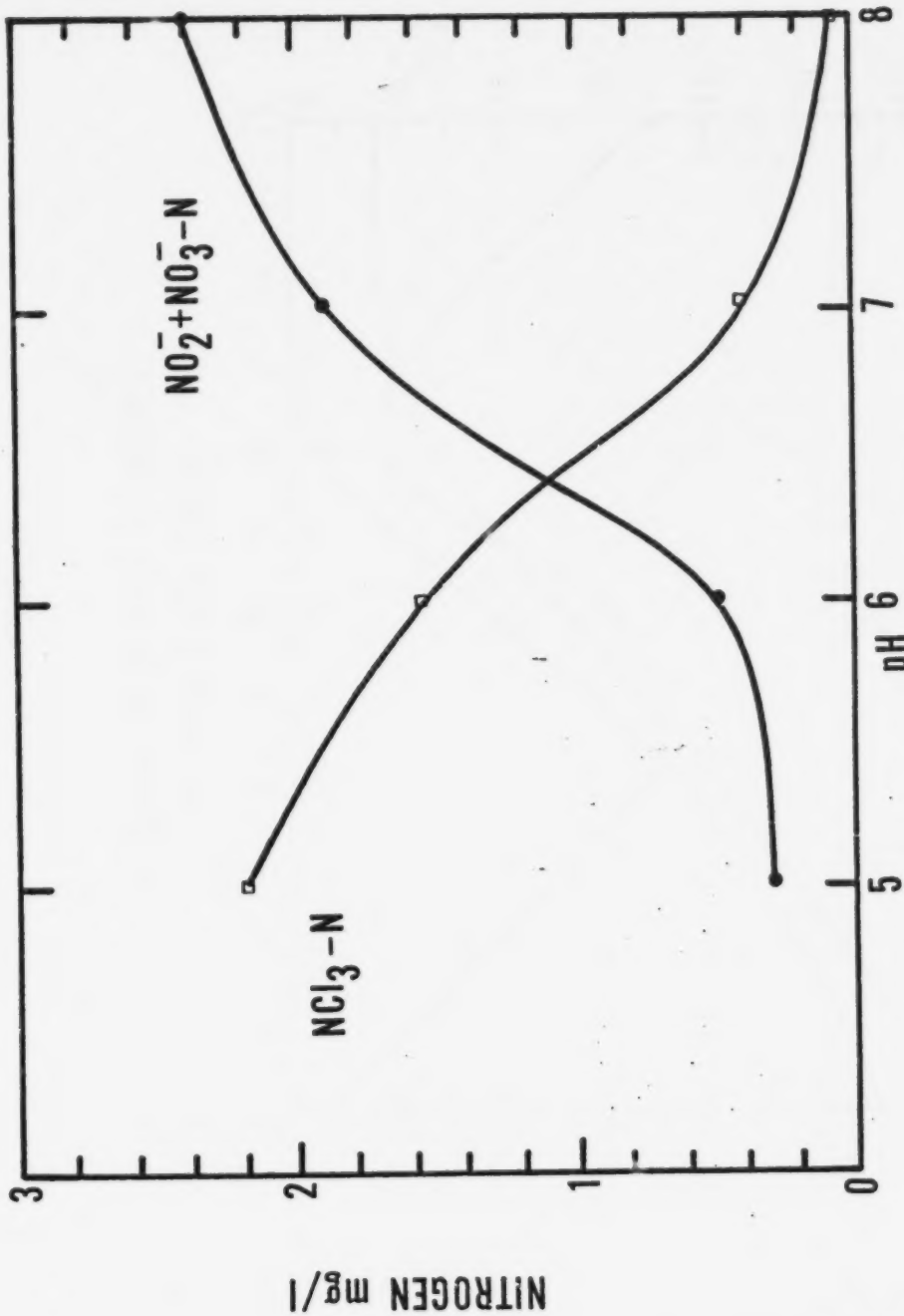
$$3\ Cl_2 + NH_4^+ \text{--------} > NCl_3 + 4\ H^+ + 3\ Cl^- \qquad (11)$$

$$4\ Cl_2 + NH_4^+ \text{--------} > NO_3^- + 10\ H^+ + 8\ Cl^- \qquad (12)$$

Reaction 11 is undesirable because it produces the odorous gas nitrogen trichloride. Reaction 12 is undesirable because nitrogen is not eliminated and the chlorine consumption is significantly higher than when nitrogen gas is the end product.

It has been found that pH is the controlling factor in maintaining reaction 10 as the predominant reaction[32]. Figure 11 illustrates that NCl_3 and NO_3 formation are kept to a minimum when the pH of the reaction is maintained between 6 and 7. Provision must be made to continuously monitor and adjust pH while this operation is being conducted as the pH tends to be depressed by reaction 10.

Figure 12 illustrates a typical breakpoint curve for nitrogen removal by chlorination of lime-clarified and filtered secondary effluent. The



EFFECT OF pH ON NO_2^-–NO3 AND NCl_3 FORMATION

FIGURE 11

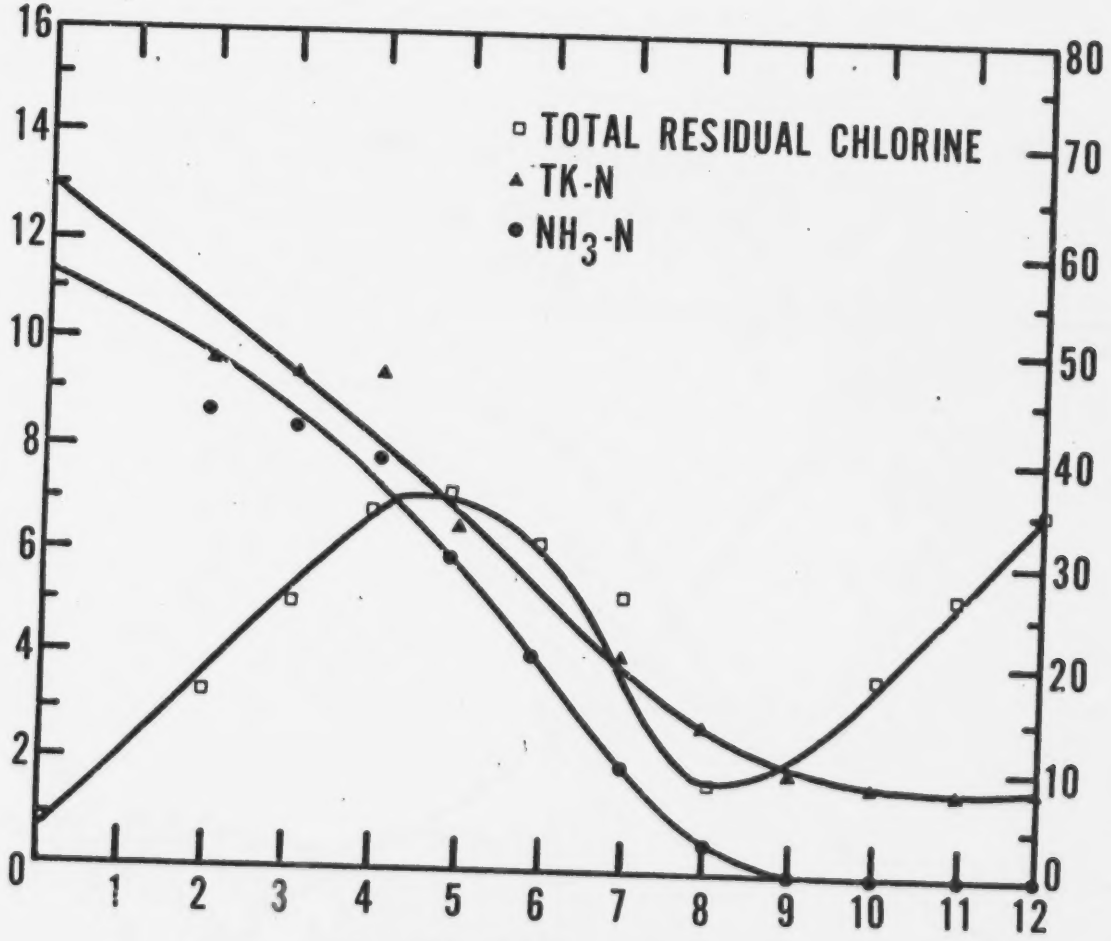

TOTAL RESIDUAL CHLORINE
TK-N
NH3-N
16
14
12
10
8
6
4
2
0
80
70
60
50
40
30
20
10
0
1
2
3
4
5
6
7
8
9
10
11
12
CL:NH3=N WT. RATIO
CHLORINATION OF LIME CLARIFIED

FIGURE 12

characteristic chlorine residual change with dose is observed with the breakpoint occurring at a dose of 8 parts of chlorine to 1 part of ammonia-N. This dose is quite close to the theoretical value of 7.6 to 1. In general, it is found that a dose close to the theoretical is sufficient. Deviations are caused by the presence of suspended solids and/or organics in wastewater. For example, secondary effluent generally requires a dose of 9 to 1, and primary effluent, a dose close to 10 to 1[30].

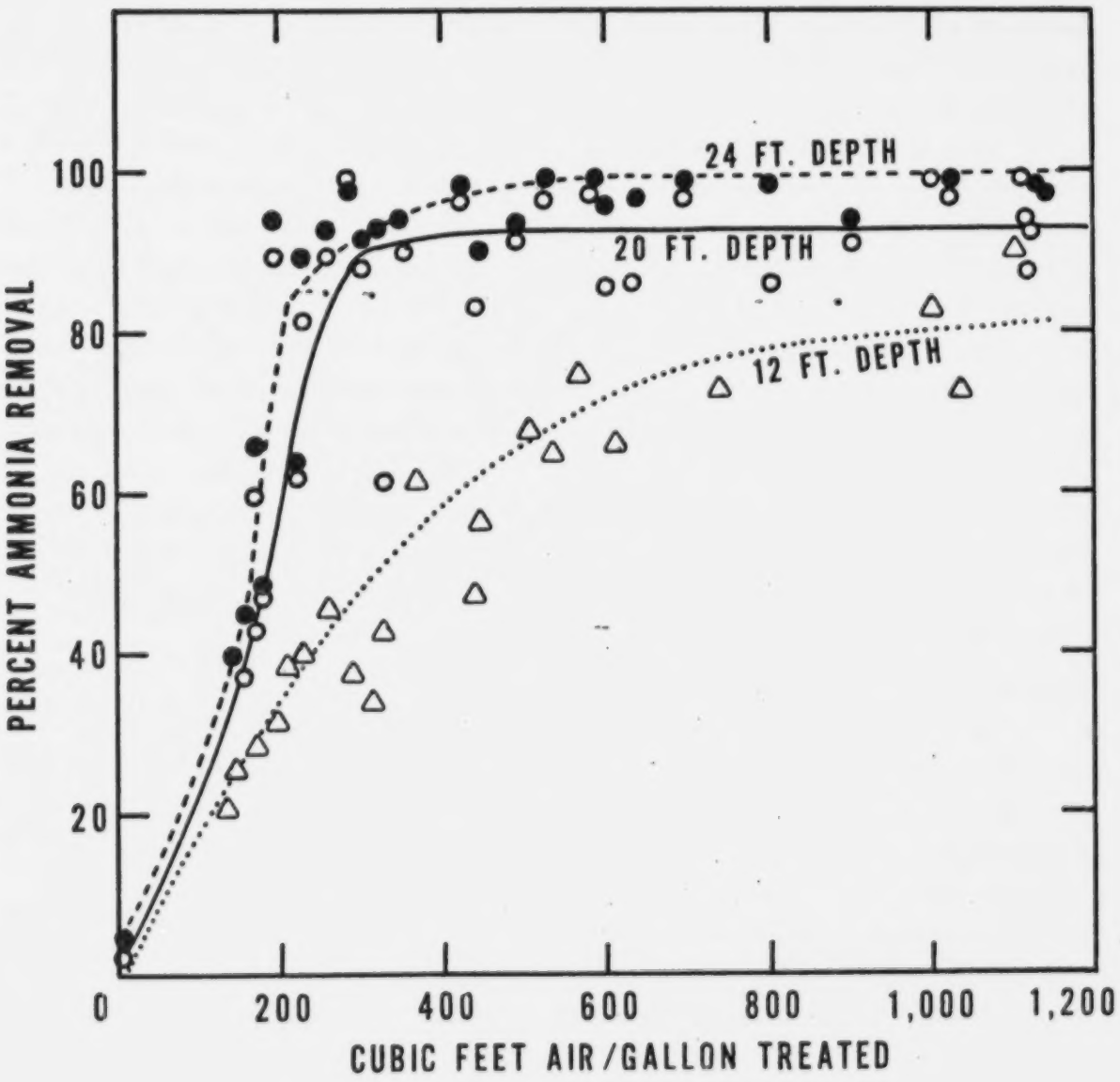
Data from studies by Bishop, et al., at Washington, D.C., and at Owosso, Michigan[34], indicate that ammonia-N concentration can be reduced to 0.1 mg/l if adequate mixing, dosing and pH control is achieved. Total removal of nitrogen is not achieved because the organic nitrogen in wastewater is not removed by the reaction with chlorine.

Two potential difficulties in this process are overdose of chlorine and mineralization of the water. The potential adverse effect on water quality of an overdose of chlorine can be prevented by the installation of a short contact time ($\approx$ 15 min) activated carbon column. Activated carbon converts residual chlorine to chloride ion. Mineralization of the wastewater as a result of the dose of 8 to 9 parts of chlorine per part of ammonia-N is a defect which cannot be avoided. This will limit the utility of this process to waters which can absorb the degree of mineralization indicated.

AMMONIA REMOVAL BY AIR STRIPPING

At neutral pH ammonia-N is present in wastewater in the soluble ionic form NH_4^+. As the pH rises, however, ammonium ion is converted to the unionized form NH_3 which is a slightly soluble gas. The solubility is a function of temperature and follows Henry's Law. Significant removals of ammonia-N from water are thus achievable by raising the pH of wastewater above 11 and stripping out the ammonia with air. The potential for the integration of air stripping with a lime precipitation process is obvious.

Slechta and Culp[35] studied this process in a pilot plant utilizing a packed tower for gas-liquid contact. Typical results illustrated in Figure 13 indicate the requirement of 400 ft^3 of air per gallon of sewage for greater than 90 percent removal. At low temperature, the greater water solubility of ammonia



PERCENT AMMONIA REMOVAL vs. AIR SUPPLY
FOR VARIOUS DEPTHS OF PACKING

FIGURE 13

requires higher air volumes for effective stripping (800 ft^3/gal at 0°C). These data were utilized to design a 3.5 MGD crossflow packed tower which was installed at Lake Tahoe. Initially, performance was good but several operational problems developed. The packed tower acted as a cooling tower and reduced the sewage temperature. This reduced the stripping efficiency below that projected and, in the extreme, resulted in icing of the tower packing. This can be overcome by heating of the air; however, because of the large volumes of air required, heating is not economically feasible. The second major problem was scaling of the tower packing with calcium carbonate precipitate. Scaling results from the contact of the calcium hydroxide in the wastewater with the carbon dioxide in the air. Scale produces a structural load on the tower which can be compensated for at an increased construction cost and interferes with gas-liquid interchange on the tower packing. At Tahoe, the scale formed was relatively soft and could be flushed from the tower; however most of the packing was inaccessible. Scaling was also experienced at the Blue Plains Pilot Plant.

Ammonia stripping is an attractive method of nitrogen removal because of its low cost, especially if all or a portion of the chemical cost to raise pH is chargeable to phosphorus removal. However, its utility will be limited by the difficulties discussed above. It may be that this procedure will be limited to the warmer areas of the country in order to escape the low temperature effect. In addition, the tower and process design will have to take into account the potential effect of scale formation. For example, Orange County, California, will soon be evaluating a tower design which provides for simple removal of sections of the packing so soft scale can be flushed away[37]. Farrell[38] has suggested the process be run at a pH of 10.5 to reduce scale formation with countercurrent contact to improve transfer efficiency.

Selective Ion Exchange

The feasibility of ion exchange for removal of ammonium ion from wastewater has until recently floundered on the greater affinity of ion exchange resins for divalent and most other monovalent ions over ammonium. While cation exchange processes removed ammonium ion, removal of all other cations in the wastewater occurred concurrently. As ammonium represents at most 10 percent of the total cation equivalents in wastewater, it is clear why ion exchange was not

economically feasible. Recently a naturally occurring zeolite clinoptilolite was found to have ion exchange properties which favored exchange of ammonium over all other cations except potassium. Fortunately, potassium is normally present at much lower concentrations than ammonium in wastewater. With this exchange material ion exchange is economically feasible for ammonium removal.

A 100,000 gpd pilot facility, consisting of two columns of clinoptilolite in series with a third as standby, was constructed by Battelle Memorial Institute and operated by EPA personnel at the Blue Plains Treatment Plant in Washington, D.C.[33]. Operation was at ≈ 4 gal/min/ft^2 downflow under pressure. The system was operated until ammonium ion started to break through the second column in series (24-30 hours of operation). The lead column was then replaced by a freshly regenerated column which was placed second in line thus assuring countercurrent contact. Regeneration was achieved by circulating a solution of lime water and sodium chloride through an exhausted column until the ammonia reaches approximately 500 mg/l. The ammonia is then air stripped from the regenerant in a small packed tower. Because the volume of air required for stripping here is much less than for direct stripping in wastewater (because the ammonia concentration is 30-50 times as high), it is economically feasible to heat the stripping air.

The work at Blue Plains[33] and other localities[39,40] has indicated that ammonia-N removal to levels of 0.1 mg/l to 0.5 mg/l are technically feasible. Overall performance reported at Blue Plains was not as good because of valve malfunctions in the regeneration system which impaired regeneration efficiency. The capacity of clinoptilolite for ammonium depends on the concentration of other cations in the wastewater and the degree of regeneration obtained. On a practical basis, a capacity of approximately one pound of ammonia-N per cubic foot of resin should be realized.

Table 18 gives the projected cost of various nitrogen control systems. Breakpoint chlorination and ion exchange are the systems which were most recently developed; consequently, cost data for these systems are the least precise. In terms of cost effectiveness and treatment efficiency it might be best to consider combinations of these processes rather than the exclusive use of one process. For example, the cost of air stripping is almost completely

TABLE 18

COSTS OF NITROGEN CONTROL

Total Cost Cents per 1000 Gallons

Plant Size / Process	1	10	100
Nitrification *	11.5	3.9	3.1
Nitrification-Denitrification *	21	8.5	5.5
Air Stripping **	7	4.0	3
Breakpoint Chlorination **		7-10	
Ion Exchange **		10-15	

* Additional Cost to Add These Stages to Activated Sludge

** Based on Ammonia-N Concentration of 20 mg/l

in capital expenditure, and the process cannot achieve high degrees of removal at low temperature. Breakpoint chlorination is very low in capital cost but is a high operating cost process. The cost of breakpoint is almost directly proportional to ammonia-N level. The advantages of a combination of these two processes is obvious.

ORGANIC CARBON REMOVAL

Organic substances are removed by most of the processes discussed in the previous sections of this paper. However, some of the organics cannot be removed by coagulation and sedimentation, nor are they amenable to biological oxidation. These have been termed refractory organics. Activities in the advanced waste treatment program were aimed at removal of refractory organics, and two methods with economic feasibility were developed: activated carbon adsorption and ozonation.

Recent studies have illustrated that these processes are just as applicable to all types of organics present in wastewater; consequently, they will be employed in the future as general purpose procedures for removal of organics in wastewater.

Activated Carbon Adsorption

The ability of activated carbon to remove soluble organics from wastewater is a result of the similarity of the surface chemistry of the activated carbon to that of the organic molecules. The ability of substances to adsorb organics is widespread; the characteristic of activated carbon which makes it unique is that it has a much higher adsorption capacity than other materials. The high capacity is the result of an extensive internal microporous structure formed during the activation process.

A generalized activated carbon treatment process is illustrated in Figure 14. The carbon and wastewater are contacted for a sufficient period of time for adsorption to take place, then they are separated. Eventually, the capacity of the activated carbon is exhausted, and it is removed from the contact vessel to a regeneration step. During regeneration some of the carbon is lost or consumed so make-up must be added.

Commercial grades of activated carbon are either granular (sizes 8 x 30 mesh, or 12 x 40 mesh) or powdered (> 300 mesh). Both forms have been found

REGENERATION & REUSE OF ACTIVATED CARBON

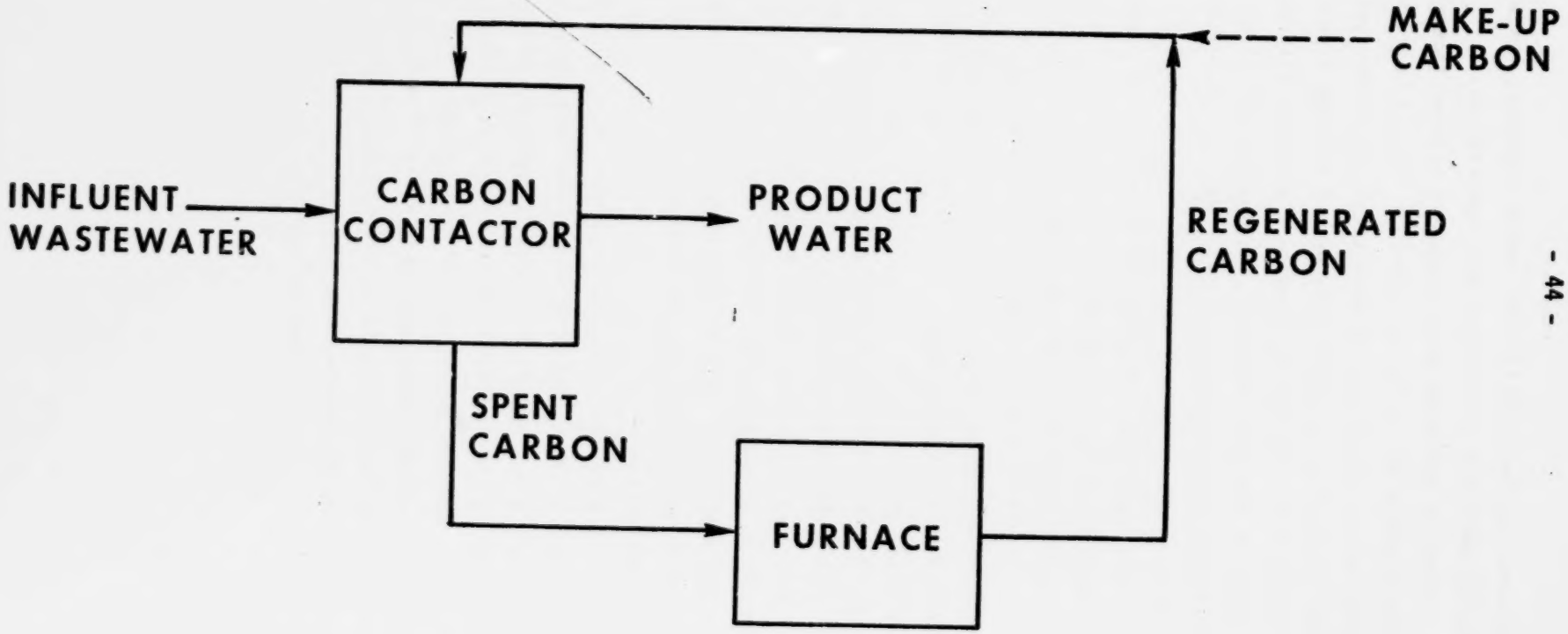


FIGURE 14

essentially equivalent in their ability to remove organics, but each type requires a different method of contacting and regeneration.

Granular carbon is contacted with sewage in columns through which the sewage flows. The columns may be either pressure vessels or gravity contactors; the former provide better operational flexibility, the latter are more economical. Flow may be upward through an expanded bed, upward through a packed bed, or downward through a packed bed. Packed bed operation provides filtration as well as adsorption but requires frequent backwash. Upflow systems allow for periodic removal of a portion of the carbon at the base, thereby providing countercurrent contact in a single vessel. However, good flow distribution is more difficult to achieve upflow than downflow. Columns can be connected in series as with the clinoptilolite system to achieve countercurrent contact. Alternately, a parallel flow arrangement can be used to achieve countercurrent contact. With the parallel flow arrangement, the starting time in service is staggered so that at any time one column is near exhaustion, one or more other columns are relatively fresh and all others are at some in-between stage. The flow from each column is blended to produce the final effluent.

Regardless of the contacting details, the fundamental system design parameter is contact time. The specific contact time to be used is a function of the effluent quality desired and the wastewater characteristics. In general, it has been found that as contact time increases, the rate at which organic removal takes place decreases. A point of essentially zero removal generally is located between 30 minutes and 60 minutes contact time. Contact time is always rated on an empty bed basis.

Regeneration of granular carbon is conducted in a multiple hearth furnace, a device which has had a long history of successful use for incineration of sewage sludge. Figure 15 illustrates a typical granular carbon regeneration scheme. Spent carbon is removed from the contactor by eduction, stored, partially dewatered, and then fed to the furnace. As the carbon moves from hearth to hearth it is dried, baked and finally reactivated. During the reactivation stage, the temperature is in the range of 1500°F - 1700°F. After regeneration, the carbon is quenched and returned to a contactor. The furnace is rated at 80 lb to 110 lb carbon per ft^2 per day. Air pollution control

ACTIVATED CARBON REGENERATION SYSTEM

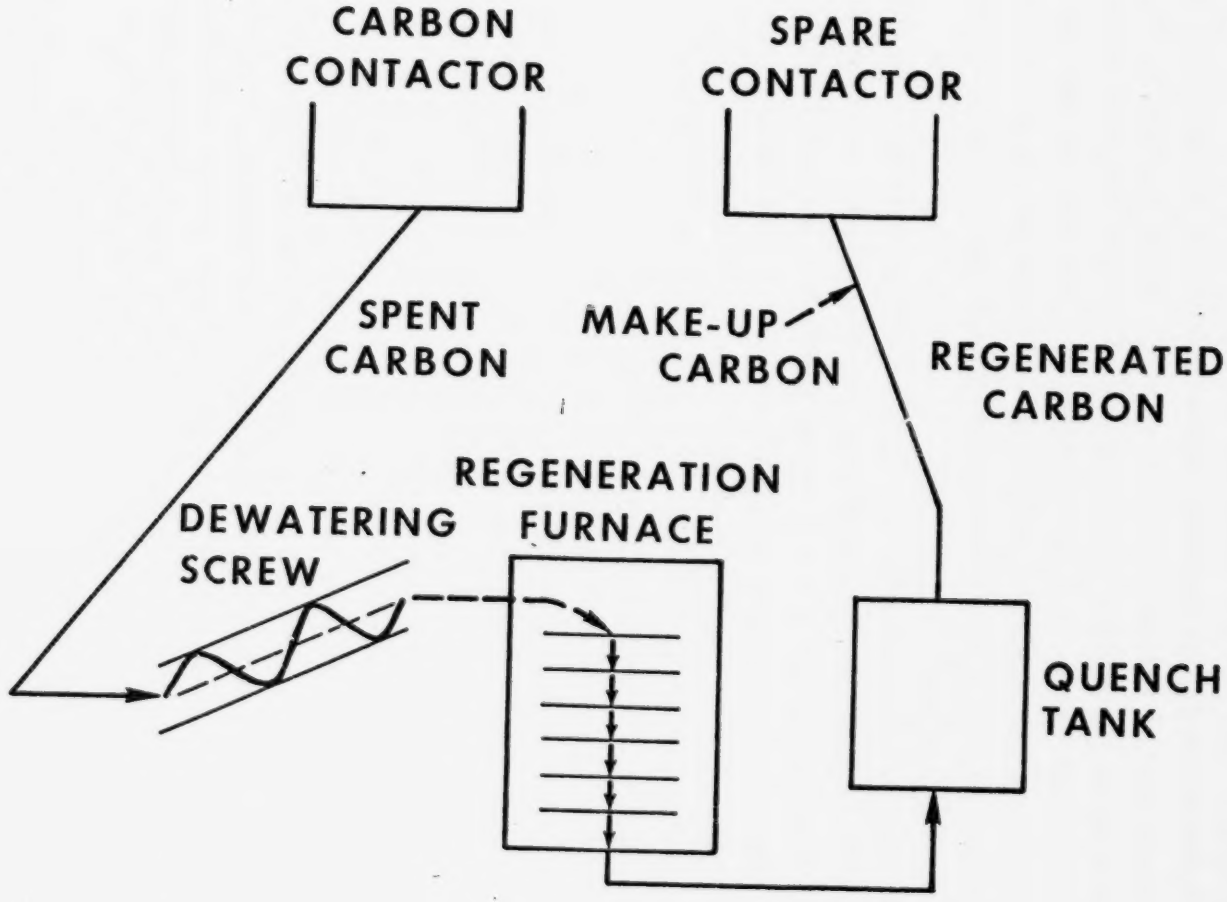
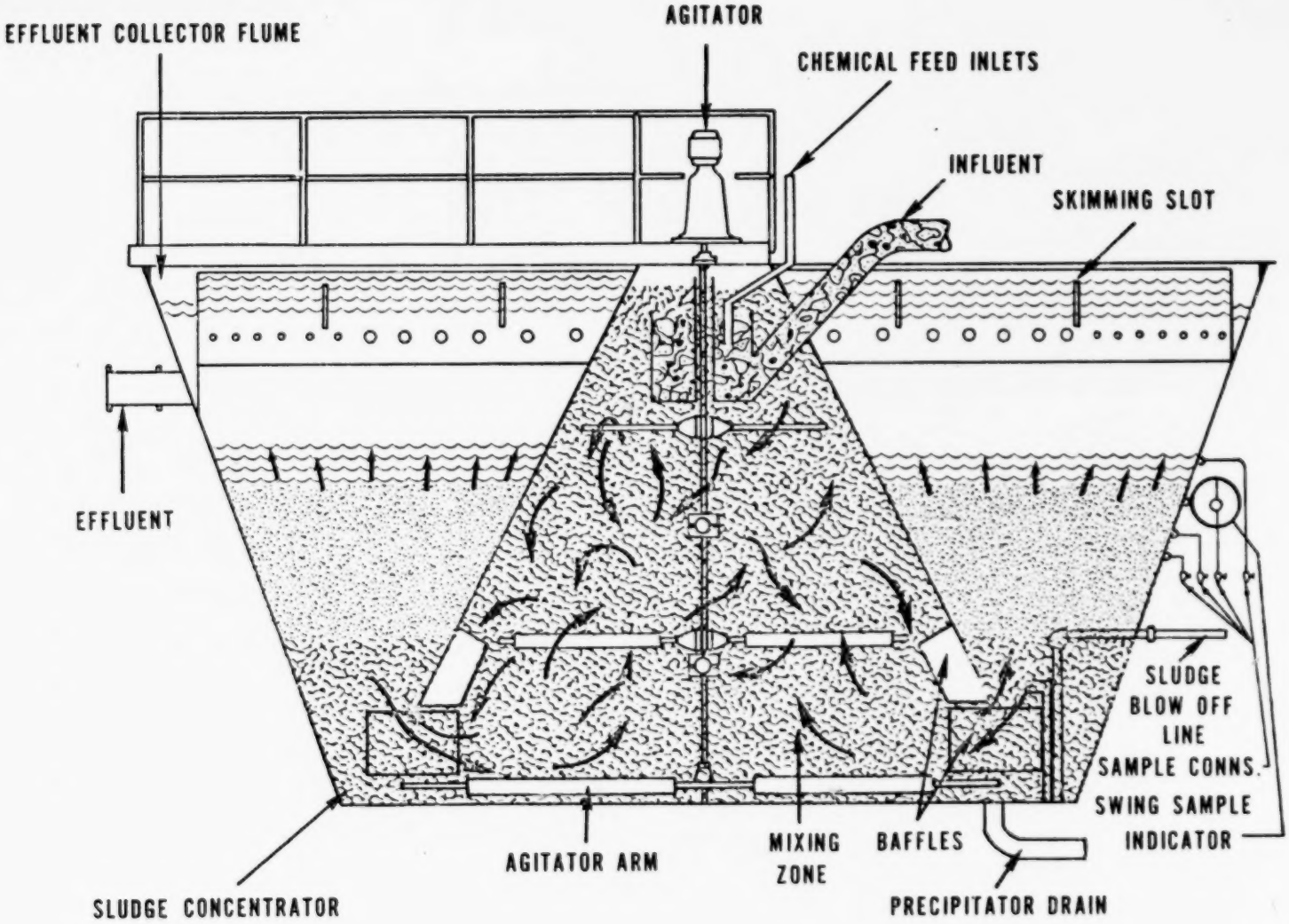


FIGURE 15

equipment, such as an after burner and wet scrubber or cyclone, are usually required to purify the exhaust. Experience with this regeneration system at essentially full scale indicates carbon regeneration loss of 5 percent to 10 percent per regeneration cycle. The losses are due to physical attrition in the regeneration loop and burnoff in the furnace[41].

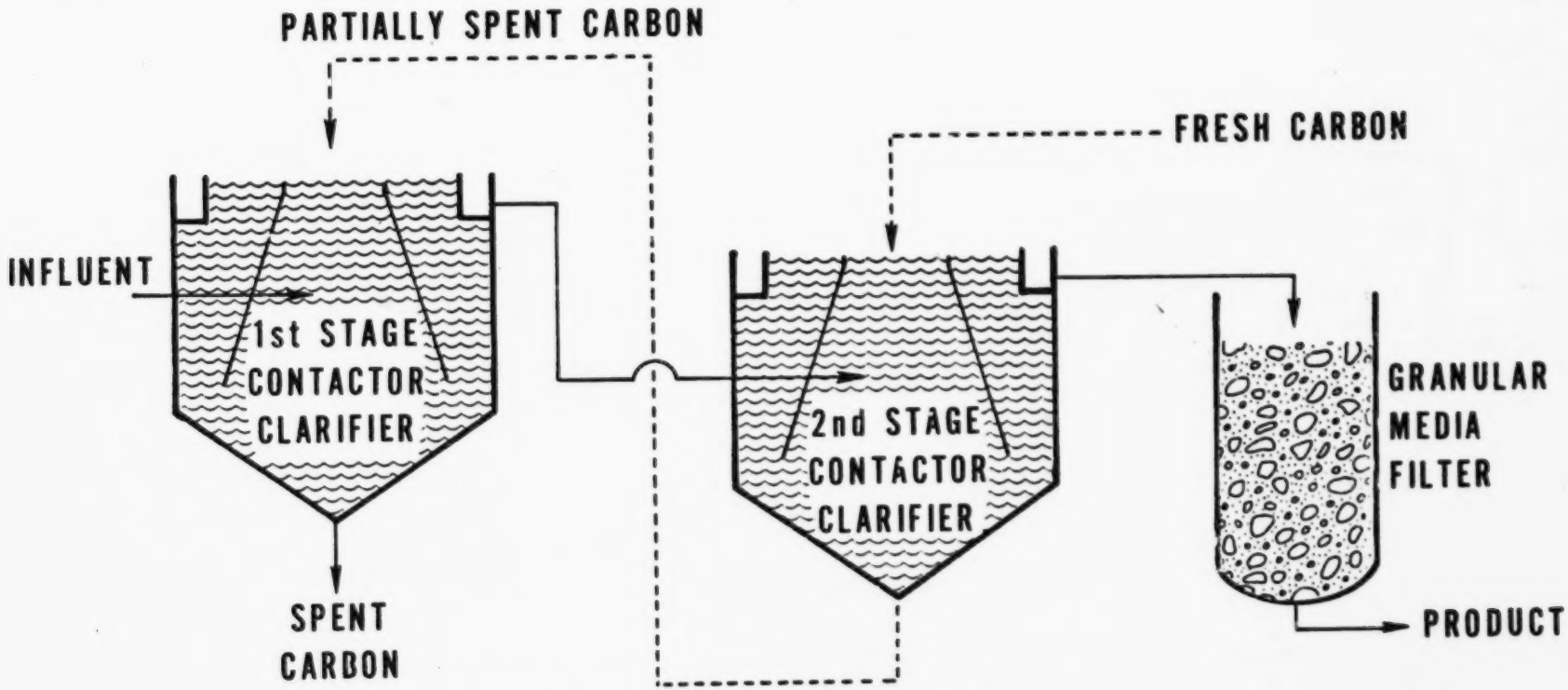
In systems utilizing powdered carbon, contact is brought about by mixing of the carbon and the wastewater in a reactor-clarifier contactor. This apparatus is illustrated in Figure 16. The carbon and wastewater are flash mixed and flocculated in the center well of the apparatus. The mixture then flows into the outer section where gravity separation of the carbon and wastewater take place. A pool of carbon in a thick slurry form is maintained at the bottom of this contactor.

In this type of contactor the carbon comes to equilibrium with the organic concentration in the effluent wastewater, whereas it comes to equilibrium with the organic concentration in the incoming liquid in a granular contactor. The organic adsorption capacity of activated carbon is proportional to the concentration of organics with which it comes to equilibrium, thus it is more imperative that countercurrent contact be achieved in powdered carbon systems than in granular systems. Figure 17 illustrates schematically a two-stage counter flow contacting system which is utilized with powdered carbon treatment. The carbon is thickened (not shown) between stages to reduce pumping and the effluent is filtered to remove residual carbon fines. Figure 18 illustrates the most advanced system for powdered carbon regeneration. The slurry containing spent carbon is thickened, dewatered in a centrifuge or on a vacuum filter and injected into a fluidized bed furnace. The regenerated carbon is reslurried and pumped back into the system. In the furnace the temperature is maintained at the same level as in the multiple hearth furnace. Contact time of a carbon particle in the furnace averages several seconds. Shorter time results in only partial regeneration, longer time results in particle incineration. Thus, tight control must be exercised with this regeneration scheme. To date small pilot studies have indicated the feasibility of this regeneration scheme with losses averaging 15 percent[42]. A larger version of this furnace is now being evaluated at Salt Lake City[43].



SOLIDS CONTACT CLARIFIER WITH SLUDGE BLANKET FILTRATION

FIGURE 16

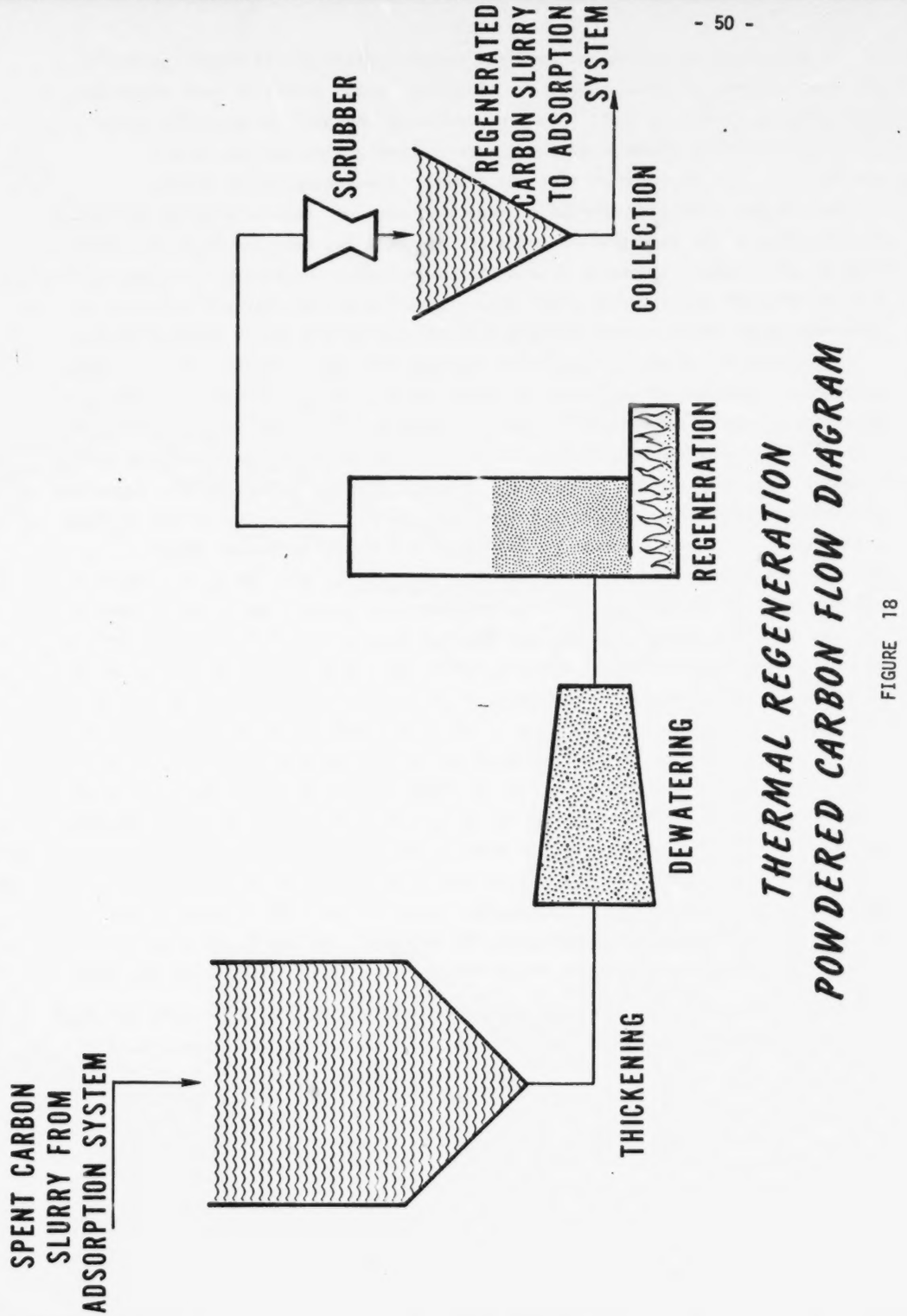


FLOW DIAGRAM

TWO-STAGE COUNTERCURRENT

POWDERED ACTIVATED CARBON SYSTEM

FIGURE 17



THERMAL REGENERATION
POWDERED CARBON FLOW DIAGRAM

FIGURE 18

Both granular carbon and powdered carbon contact are excellent processes for organics removal from wastewater. Powdered carbon processes have advantages over granular carbon in that: powdered carbon is cheaper than granular carbon (10¢/lb vs 30¢/lb); powdered carbon plants require a much smaller carbon inventory and can be dosed to meet the demand. However, granular carbon exhibits higher practical adsorption capacity, does not require complex dewatering procedures prior to regeneration and most important has been successfully regenerated at full scale. Economics demand that regeneration be incorporated into all but the smallest scale carbon adsorption plants. Thus, an engineer designing a treatment plant using carbon adsorption would most likely choose granular carbon.

Extensive processing experience has been obtained with the use of activated carbon for treatment of wastewater after biological treatment (tertiary application). Data from Pomona[44], Lebanon[45], Colorado Springs[46], South Lake Tahoe[4], and Tucson[47] are summarized in Table 19. It can be seen that organic removal in the range of 70 percent to 80 percent was obtained. This is typical of the capability of activated carbon for organic removal from sewage. Data on adsorption capacity indicates a considerable spread, 0.25 to 0.87 lbs COD/lb activated carbon[41]. This considerable spread occurs because carbon capacity is a function of desired effluent quality and influent organic concentration. Most significant, however, is that in all cases the capacity was significantly higher than that projected by an adsorption isotherm test. This observation was recently confirmed at a number of plants in Pennsylvania[48]. Biological activity on the carbon is the primary explanation of the significantly higher capacity obtained. It is theorized that partial regeneration in situ is accomplished by biological action. Naturally, this has had a favorable impact on the economics of carbon adsorption. Table 20 summarizes economics of granular carbon adsorption. It can be seen that regeneration and make-up carbon account for about 25 percent of the process cost.

The process design for carbon treatment has had to be adjusted to take account of biological action. For example, only a normal upflow backwash was provided in early granular carbon contactor design. This was found to be inadequate to insure cleaning of the carbon column so air scour and surface wash are frequently added. In addition, more frequent backwash (at least once per day) must be planned for in design. Under heavy load, carbon columns become septic

TABLE 19

ACTIVATED CARBON TREATMENT OF WASTEWATER TERTIARY APPLICATION

Location	Carbon	Organics, mg/l		% Removal	Organics Measurement
		In	Out		
Pomona	Granular	43	10	77	COD
Colorado Springs	Granular	43	13	70	TOC
S. Lake Tahoe	Granular	12	3	75	TOC
Lebanon	Powdered	20	4	80	TOC
Tucson	Powdered	27	7.5	72.5	COD

TABLE 20

ECONOMICS OF GRANULAR CARBON TREATMENT

Capitol Cost Breakdown		Operating Cost Breakdown		Total Cost ¢/1000 gallons	
Function	%	Function	%		
Carbon Inventory	20	Makeup carbon	20	1 MGD	30
Contactors	41	Labor	12	10 MGD	10
Regeneration	12	Power	12	100 MGD	4
Pumps	27	Fuel	2		
		Backwash water	4		
		Maintenance	10		
		Capital Recovery	40		

and hydrogen sulfide is produced. Aeration of the feed may be required or extra-chlorination to oxidize the effluent sulfide may be needed. Under heavy load, expanded bed operation becomes superior to packed beds because clogging is less of a problem and aeration is easier to accomplish.

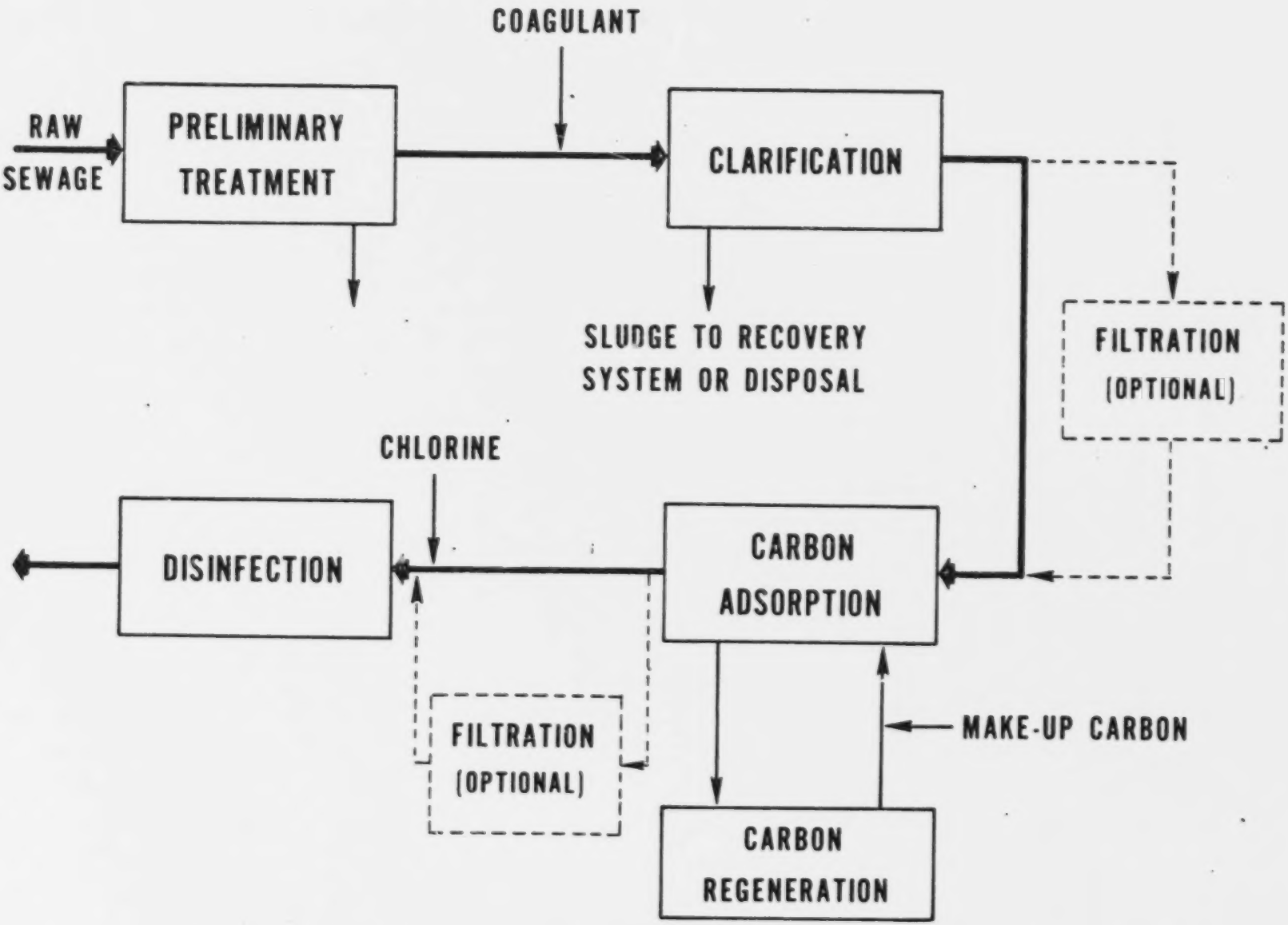
Powdered carbon contact design has also been affected by the realization that biological action may occur. It has recently been illustrated that when extensive biological action is encouraged, only one carbon contacting stage is required rather than two to achieve high adsorption capacity[43].

Physical-Chemical Treatment

Physical-chemical treatment of raw waste water or more accurately chemical clarification - carbon adsorption was first evaluated as a treatment technique over three decades ago. It was found to produce a good quality effluent but at a cost greater than conventional processing. The recent upgrading of water quality standards has brought this treatment scheme closer to economic viability because it can produce an effluent superior to conventional processing. In addition, the developments discussed above in carbon adsorption techniques have tended to lower the cost of physical-chemical treatment. Finally, removals achieved by chemical clarification have been higher than anticipated which has also had a favorable effect on physical-chemical treatment costs.

Figure 19 is a general diagram of the clarification carbon treatment system. Following the standard type of preliminary treatment, the waste is dosed with chemical sufficient to achieve the desired level of suspended solids and/or organics and/or phosphorus removal required. The same chemicals are used as for phosphorus removal in the primary. Significant removals of organics, suspended solids and phosphorus are obtained by chemical clarification as illustrated in Table 21[49]. Especially significant is the organic removal which is higher than would be expected from complete removal of suspended solids. This extra removal is due to coagulation and possible chemical adsorption of colloidal and soluble organics. Up to 50 percent of the "soluble" organics in sewage may be removable by chemical coagulation[50].

After clarification is complete, the wastewater is passed to the carbon adsorption step for completion of the removal of soluble organics. Table 22 gives data from a number of plants which indicate that very good quality effluents



FLOW DIAGRAM OF A PHYSICAL-CHEMICAL TREATMENT SYSTEM

FIGURE 19

TABLE 21

ACHIEVEMENTS OF CHEMICAL CLARIFICATION

PLANT	CHEMICAL	ORGANIC REMOVAL %	SS REMOVAL %	P REMOVAL %
Ewing-Lawrence	170 mg/l $FeCl_3$	80	95	90
New Rochelle (ZM)	Lime pH 11.5	80	98	98
Westgate, VA.	125 mg/l $FeCl_3$	70	-	-
Salt Lake City	80-100 mg/l $FeCl_3$	75	-	80
Blue Plains	Lime pH 11.5	80	90	95

TABLE 22

ACTIVATED CARBON PERFORMANCE IN
PHYSICAL-CHEMICAL TREATMENT PLANTS

PLANT	EFFLUENT ORGANICS mg/l	CARBON CAPACITY lbs TOC / lb A.C.	CAPACITY lbs COD / lb A.C.
Blue Plains (G)	TOC = 6	0.15	0.41
Lebanon (G)	TOC = 6	0.22	0.50
Ewing-Lawrence (G)	TOC = 3-5	0.3	-
New Rochelle (G)	COD = 8	-	0.6
Owosso (G)	COD = 25	-	0.65
Salt Lake City (P)	COD = 22	-	0.36

(G) - Granular Carbon
(P) - Powdered Carbon

can be obtained as well as high carbon adsorption capacity[34,43,49]. As indicated previously, this is the result of biological action on the carbon. In physical-chemical treatment systems, the carbon is subjected to a heavier load than in tertiary plants, thus extra care in design must be taken to insure that the disadvantageous features of biological action on carbon can be mitigated.

Filtration is presented as an optional step in Figure 19, but it would be wise to include it in the treatment scheme. Filtration acts as the safety factor in the solids removal step; therefore, its use can bring a high degree of reliability to the treatment scheme. In addition, it can be used to prevent an overload of solids on the carbon which could foster excessive biological action. The positioning of the filter prior to or after the carbon contact is dictated by the method of carbon contact. Expanded bed and powdered carbon systems usually require filtration after the carbon step. Packed bed systems will benefit from filtration before the carbon. One plant has been designed with filtration before and after carbon contact. Several plants have been designed without a filtration step[48].

Tables 23, 24, and Figure 20[49] provide information on the performance of three physical-chemical treatment pilot plants. These data illustrate removals at various stages of treatment for a variety of pollutants. The Blue Plains data include nitrogen removal by clinoptilolite.

Cost estimates for physical-chemical treatment are presented in Table 25. These should be compared to the cost of conventional treatment with phosphorus removal added, i.e., data in Tables 2 and 13, as treatment performance of both systems is comparable.

Table 26 lists the features which physical-chemical treatment has with respect to conventional treatment systems. Because of these features and a favorable cost comparison a number of communities are planning physical-chemical treatment systems. These are listed in Table 27.

Ozone Oxidation

In addition to carbon adsorption, another physical-chemical method of organics removal, ozonation, has been evaluated and found worthy of development. However, this procedure is several years behind activated carbon in

TABLE 23

POWDERED CARBON PILOT PLANT OPERATING CONDITIONS

Flow Rate 50 gal/min

CHEMICAL 425 mg/1 LIME to pH 10.8

CARBON 150 mg/1 + 0.4 mg/1 polymer

RESULTS

	COD	BOD	SS	P
RAW SEWAGE	222	144	200	7.3
CLARIFIED EFFLUENT	65	47	28	1.4
FINAL EFFLUENT	35	13	7	0.4

TABLE 24

ROCKY RIVER WASTE TREATMENT PLANT

CLARIFICATION-CARBON PROCESS

	RAW	POLYMER CLARIFICATION	CARBON CONTACT TIME, minutes			% REMOVED
			14	23.4	32.6	
SUSPENDED SOLIDS, mg/1	107	65	13	15	7	93.3
BOD, mg/1	118	57	21	11	8	93.3
COD, mg/1	235	177	67	50	44	81.3

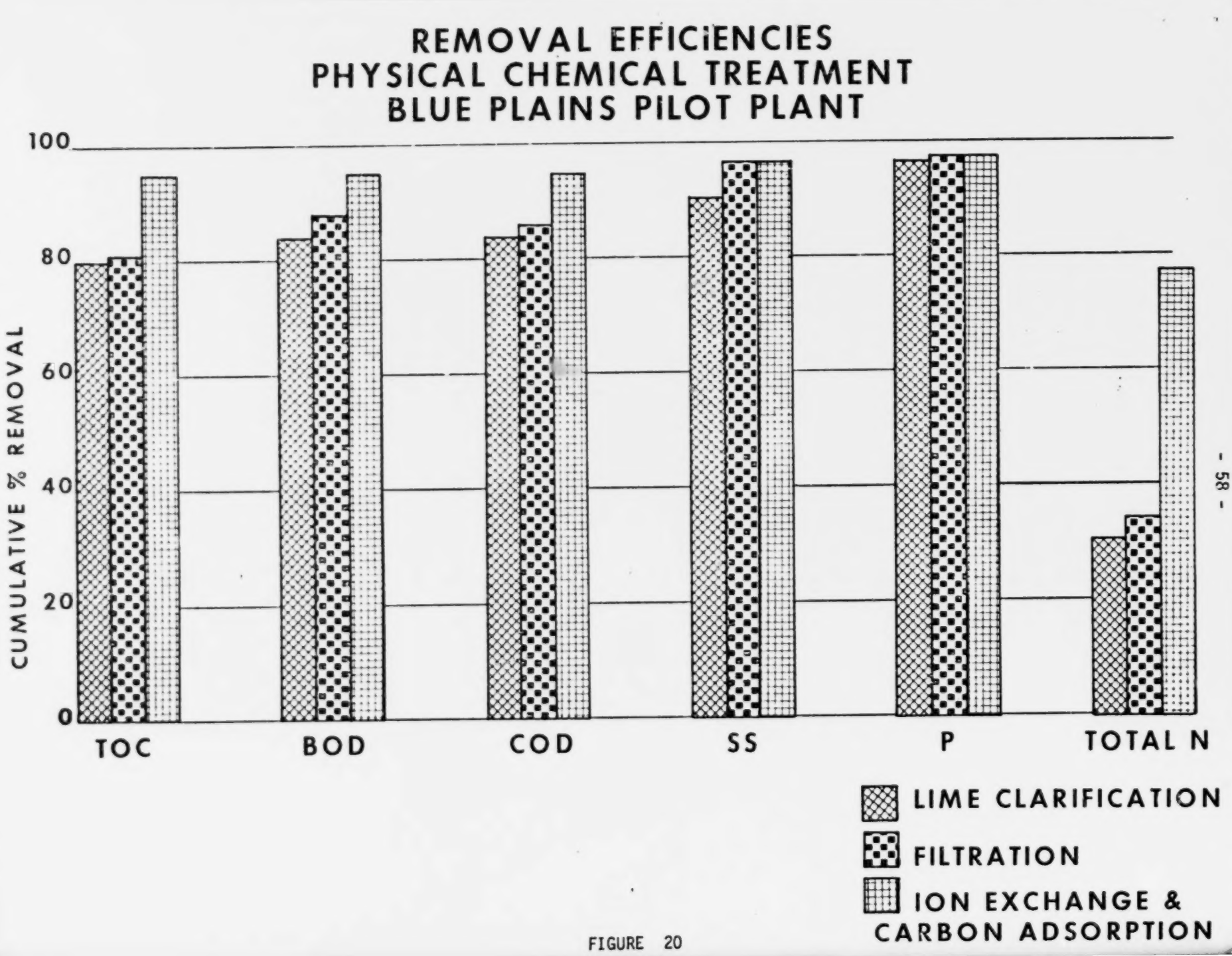

REMOVAL EFFICIENCIES
PHYSICAL CHEMICAL TREATMENT
BLUE PLAINS PILOT PLANT
100
80
60
40
20
0
CUMULATIVE % REMOVAL
TOC
BOD
COD
SS
P
TOTAL N
LIME CLARIFICATION
FILTRATION
ION EXCHANGE &
CARBON ADSORPTION

FIGURE 20

TABLE 25

PRELIMINARY COST ESTIMATE PHYSICAL-CHEMICAL TREATMENT

Total Amortization + O&M, cents per 1000 gallons

PLANT SIZE, MGD	5	10	100
Chemical Clarification*	9.5-13.5	7.3-9.6	4.0-5.3
Carbon Adsorption	11.5-18.0	9.1-13.5	4.5-7.8
Filtration	2.9-4.5	2.1-3.3	1.0-1.4
TOTAL	23.9-36.0	18.5-26.4	9.5-14.5

*Two-stage lime recalcination of sludge.

TABLE 26

ADVANTAGES OF
PHYSICAL-CHEMICAL TREATMENT
vs.
CONVENTIONAL PRIMARY + SECONDARY

1. LESS AREA REQUIREMENT - 1/2 to 1/4
2. LOWER SENSITIVITY TO DIURNAL VARIATION
3. NOT AFFECTED BY TOXIC SUBSTANCES
4. POTENTIAL FOR SIGNIFICANT HEAVY METAL REMOVAL
5. SUPERIOR REMOVAL OF P COMPOUNDS
6. GREATER FLEXIBILITY IN DESIGN AND OPERATION
7. SUPERIOR ORGANIC REMOVAL

TABLE 27

PHYSICAL-CHEMICAL TREATMENT PLANTS

SITE	CAPACITY	STATUS
ROCKY RIVER, OHIO	10	BIDS TAKEN
PAINESVILLE, OHIO	5	DESIGN
CLEVELAND, OHIO	10	DESIGN
FITCHBURG, MASSACHUSETTS	15	DESIGN
WATERFORD, NEW YORK	2	PLANNED
CORTLAND, NEW YORK	10	PLANNED
CLAY, NEW YORK	10	PLANNED
NIAGARA FALLS, NEW YORK	60	PLANNED
GARLAND, TEXAS	30	DESIGN
OWOSSO, MICHIGAN	6	DESIGN

development because so much required information has not yet been obtained.

Ozone is a powerful oxidant which is generated by passing a properly conditioned air stream between electrodes, one of which produces a corona discharge. It must be generated on site and used immediately. Ozone is only poorly soluble in water so that special contacting techniques must be used to insure optimum utilization. Based on preliminary work by Airco[52], a 50,000 gallon per day plant was installed and operated at Washington, D.C.[53]. In this plant, ozonation of effluents from a variety of treatment processes was evaluated. It was found that 50 percent to 70 percent of the COD was oxidized rather easily but oxidation of the remainder was difficult to achieve. High pH values increased ozone utilization efficiency and stimulated nitrogen oxidation to nitrate. Ozone consumption averaged 3 lbs ozone per lb of COD removed. At this level of consumption, it is economically feasible to use ozone only as a polishing technique after extensive treatment by other organic removal methods. It is anticipated that ozone will also find utility in the future for disinfection.

Pure Oxygen Activated Sludge

The last improvement in procedures for removing organics from wastewater which will be discussed is the application of pure oxygen in the activated sludge process. The use of pure oxygen does not permit activated sludge to go beyond its inherent capability but allows a closer approach to this limit within an acceptable economic framework.

Early work in the biological treatment field pointed out the advantages of pure oxygen over air as the source of oxygen in activated sludge[54]. However, the cost of the pure oxygen was quite high and a feasible system to achieve high degrees of utilization was not available, so the concept lay dormant. Over the last decade, the cost of pure oxygen has decreased and a system was devised which could achieve high utilization efficiency. Figure 21 illustrates the UNOX* system developed by Union Carbide. The key features of this system are: the use of covered reactors to prevent loss of oxygen; a baffled reactor to prevent short-circuiting and recompression and recycle of gas in each chamber. In all other respects, the system is virtually identical to a conventional activated sludge.

The performance of this system was evaluated in parallel with a conventional activated sludge plant in Batavia, N.Y.[55]. The study was conducted in three phases. In Phase I, one-half of the activated sludge tankage of the plant was run with pure oxygen, the other half with air. In Phase II, all flow was routed through the half with pure oxygen feed. In Phase III, one-fourth of the plant was on pure oxygen, one-fourth was on air and the other half of the plant was out-of-service. Pertinent data from this study are presented in Table 28.

These data indicate the superiority of the pure oxygen system over conventional aeration. Under much heavier volumetric loading rates, the oxygen system produced a better effluent than the air system. Key to the improved performance is the development of a sludge which can settle quite well even at high concentration. The pure oxygen system developed higher sludge concentrations in the aeration tank; consequently, the actual load per organism (F/M) was lower than in the air system. In effect, the pure oxygen system could operate at a greater sludge age than the conventional system. Theory indicates that the greater the sludge age the better the BOD removal in a biological waste treatment system.

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*Mention of manufacturer does not constitute endorsement by Environmental Protection Agency.

SCHEMATIC DIAGRAM OF MULTI-STAGE OXYGENATION SYSTEM

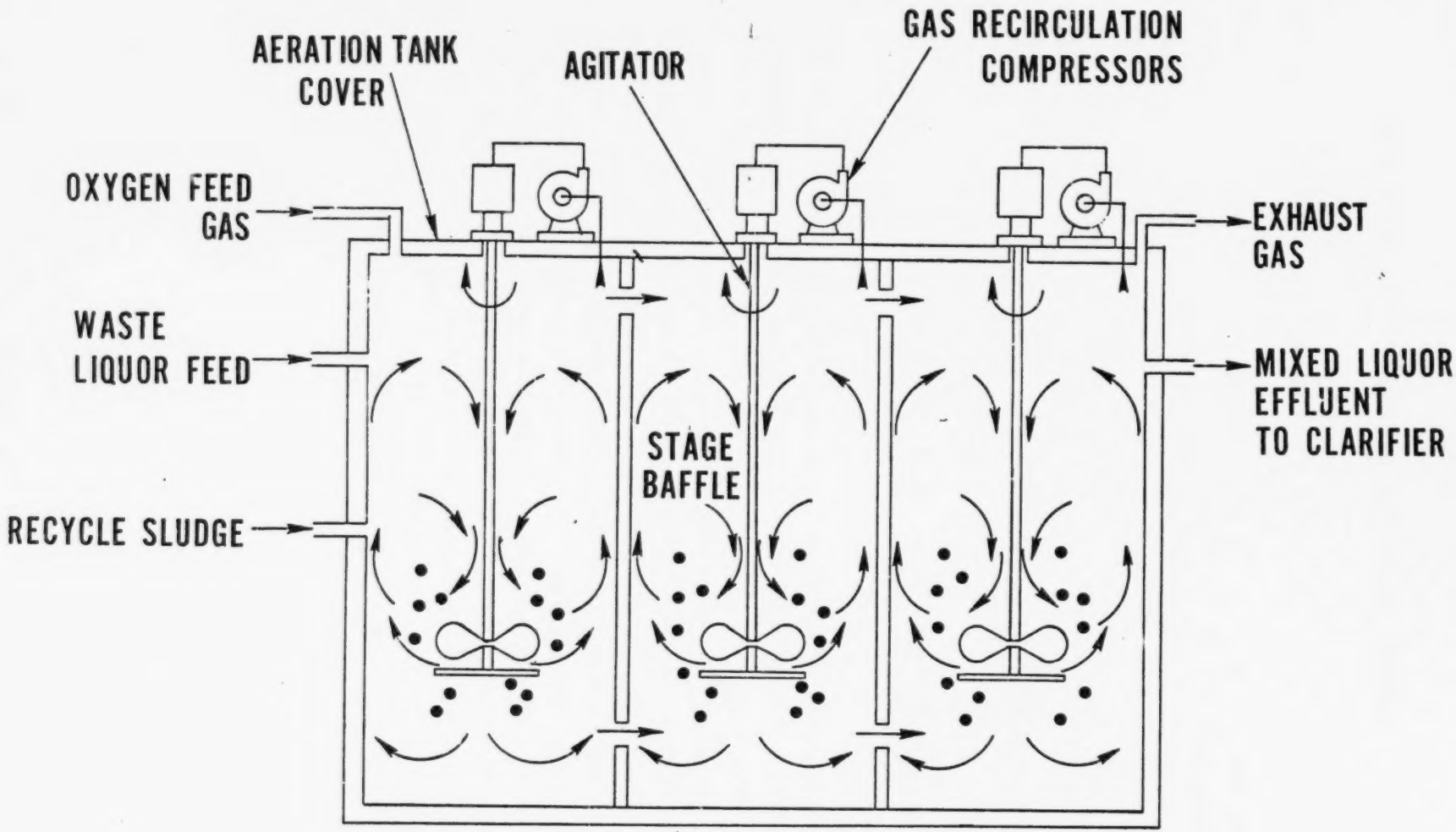

FIGURE 21

TABLE 28

BATAVIA, N. Y. PURE OXYGEN ACTIVATED SLUDGE STUDY

	PHASE I		PHASE II	PHASE III	
	Air	Pure O_2	Pure O_2	Air	Pure O_2
Sewage Flow, MGD	1.97	1.91	2.53	1.29	1.40
Aeration Time, hrs.	3.5	3.4	1.2	2.6	2.0
BOD, in	159	159	220	262	262
BOD, out	16	11	23	30	14
MLSS	2440	3060	6980	3640	6190
SVI	76	64	36	63	49
Volumetric Loading, $\frac{\text{lbs BOD}}{1000\ \text{ft}^3}$	60	58	212	129	145
F/M, $\frac{\text{lbs BOD}}{\text{lbs MLSS}}$	0.57	0.41	0.79	0.84	0.55
Excess Sludge, $\frac{\text{lbs VSS}}{\text{lb BOD}}$	0.87	0.48	0.41	0.99	0.13
O_2 Utilization, %	-	95.5	92.7	-	91.4

The economic advantage of a pure oxygen system is attributable to the higher volumetric loadings allowable and the lower excess sludge quantities produced both in mass and volume. As Figure 22 indicates, these are sufficient to more than offset the extra cost of supplying pure oxygen rather than air.

Additional studies of this system have been carried out at Blue Plains in Washington, D.C.[56]. The results of this study are illustrated in Table 29. These data confirm most of the observations from Batavia, i.e., low sludge yield, high sludge age, and excellent BOD removal with a short aeration period (1.95 hours). During the winter, however, sludge sedimentation deteriorated seriously. This problem can be overcome by adequate design of the final sedimentation basin. Final clarifier overflow rate should be kept below 900 gal/day/ft^2, rather than as high as originally projected 1500 gal/day/ft^2 [55].

Table 30 provides a list of large-scale plants in which pure oxygen, aeration activated sludge will be installed.

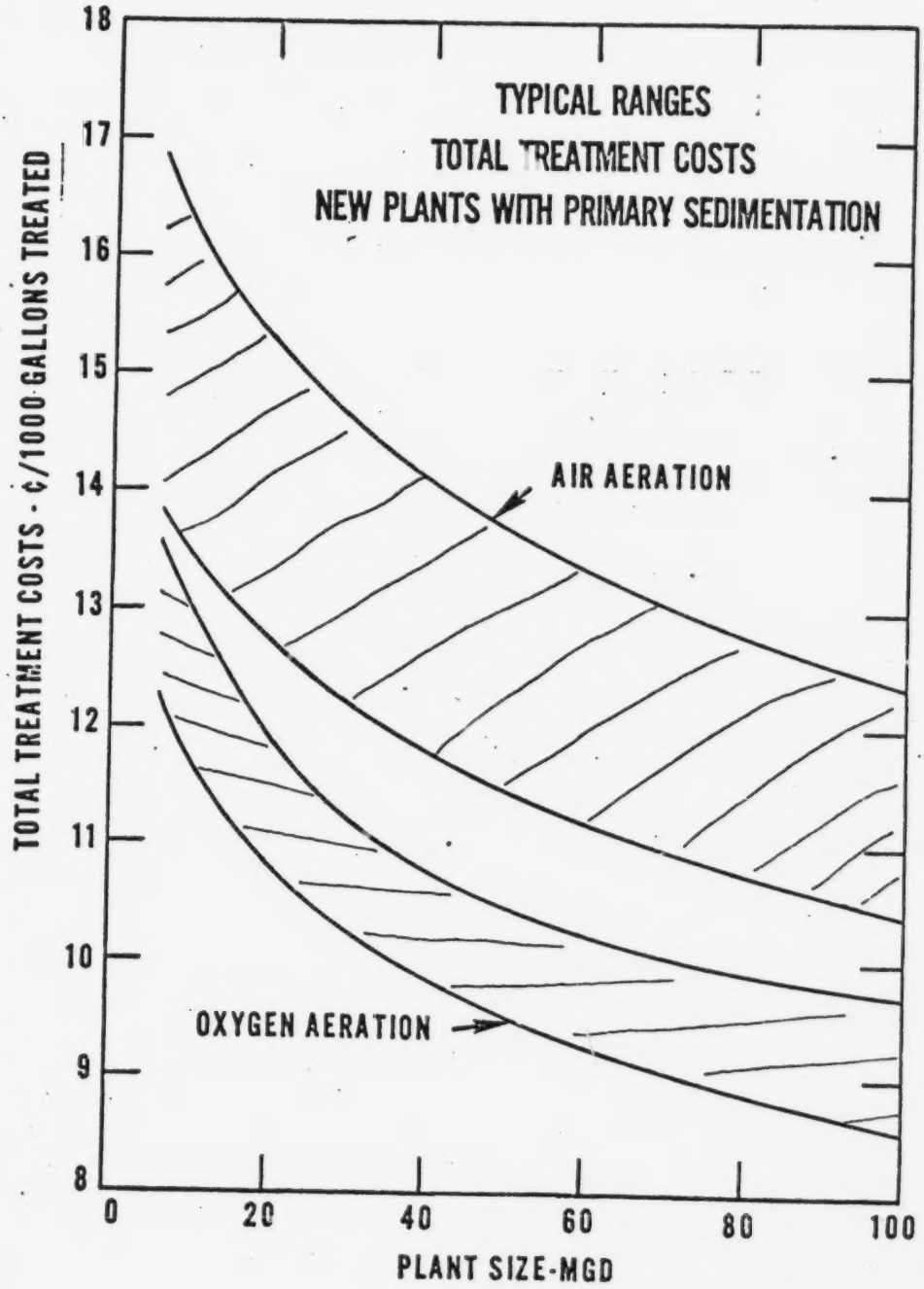
TYPICAL RANGES
TOTAL TREATMENT COSTS
NEW PLANTS WITH PRIMARY SEDIMENTATION
AIR AERATION
OXYGEN AERATION
TOTAL TREATMENT COSTS - ¢/1000 GALLONS TREATED
PLANT SIZE-MGD
18
17
16
15
14
13
12
11
10
9
8
0
20
40
60
80
100

FIGURE 22

TABLE 29

SUMMARY OF REMOVAL EFFICIENCY
RESULTS WITH OXYGEN AT WASHINGTON, D. C.

PARAMETER	PRIMARY	SECONDARY EFFLUENT
BOD, mg/l	87-132	11-32 (2-3 soluble)
COD, mg/l	244-284	45-99
TOC, mg/l	65-106	14-26
S. S., mg/l	-	24-58

*1.95 Hours Detention

TABLE 30

FUTURE PURE OXYGEN ACTIVATED SLUDGE PLANTS

LOCATION	SIZE, MGD
DETROIT, MICHIGAN	300
NEW YORK, N.Y.	20
WINNIPEG, CANADA	15
NEW ROCHELLE, N. Y.	15
SPEEDWAY, INDIANA	12
YORK, PENNSYLVANIA	8

DISSOLVED INORGANICS REMOVAL

In the process of its domestic or industrial use an increment of dissolved mineral matter is added to water. The magnitude varies from 100 mg/l to 500 mg/l per use depending generally on local conditions. Extensive reuse of wastewater even for industrial use will require at least partial demineralization of wastewater. A number of demineralization processes were evaluated in the advanced waste treatment programs and only three, electrodialysis, ion exchange, and reverse osmosis were promising enough for additional studies.

Ion Exchange

At present, ion exchange shows much promise for removal of dissolved inorganics. Cost of operation is lowest, especially at the salt levels anticipated in wastewater and the technology of ion exchange at full scale is proven. A four-bed strong acid-weak base system was tested at Pomona, California[56] with a carbon treated feed. Total dissolved salts were reduced to 80 mg/l from 600 mg/l. Because of the very low TDS in the product blending of the treated and untreated water is likely with attendant reduced overall cost.

Ion exchange systems require regeneration with solutions much higher in TDS than the original feed yielding a difficult problem of brine disposal. In order to reduce this problem, special combinations of resins have been devised. One such system is the DESAL* process. In this scheme, flow is first through a weak base anion where all anions are converted to bicarbonate. Flow then is through a weak acid cation column which exchanges all cations for hydrogen ion. The hydrogen and bicarbonate ions react to form CO_2 and water, and the demineralization is essentially complete. The effluent is, however, then run through a weak base anion in the hydroxide form. The effluent converts the hydroxide and CO_2 to bicarbonate on the resin. Thus, during the next cycle the third column can function in the first position without any regeneration required.

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*Mention of manufacturer does not constitute endorsement by Environmental Protection Agency.

Electrodialysis

In sewage application, electrodialysis suffers severe limitations. The anion plates foul quite easily, drastically reducing treatment performance unless the feed has received extensive pretreatment[57]. In addition, as the sewage is demineralized its electrical resistance increases. This limits the degree of economically feasible demineralization which can be obtained. It is difficult to reduce the TDS of water much below 300-400 mg/l with electrodialysis. For these reasons electrodialysis is no longer considered a leading candidate process.

Reverse Osmosis

At present, reverse osmosis is the most expensive of the three processes discussed for demineralization. However, its potential is considerably superior to that of the other two. This process is indeed in its infancy as the first working membranes were developed only 15 years ago. In addition, membranes which will desalt water will remove virtually every other pollutant from wastewater. Thus, reverse osmosis may become an all purpose pollution control system.

In this process water is forced to flow through a semi-permeable membrane (permeable to water but not to salts) by application of high pressure. Pressures of several hundred psi are required to achieve fluxes of the order of 10 gallons per ft^2 per day. The degree of demineralization can approach 99+ percent and increases with the pressure applied.

The heart of this process is the membrane. At present, the best membranes are cast from a mix of cellulose acetate, acetone, formamide and magnesium perchlorate. Special techniques form a membrane with a thin (1 micron) skin which provides the desalting surface and a thick (100 microns) porous sublayer which supplies a structural backing. This membrane is compressible, hydrolyzes at low and high pH and is easily damaged. Work is being conducted to develop new, tougher membrane materials.

Even if new membrane materials are found, the engineering problems of this process will remain unchanged. These problems are a structural support system to absorb the several hundred psi pressure drop across the membrane and control of fouling on membrane surfaces. The support system must not only support the membrane but should provide high membrane area per unit volume.

Three configurations are in use, tubular, spiral-wound, and hollow fine fiber. The tubular system has the membrane positioned along the inner wall of a porous 1/2 in. diameter tube. The spiral-wound module uses a stack of flat membranes separated by spacers and rolled into a jelly-roll form. The hollow fiber uses microscopic fibers of the membrane which are in essence thick-walled microcylinders. Here, pressure is applied outside the fiber and water permeates into the hollow core. As a rough guide, membrane area per unit volume is 20 ft^2/ft^3 for tubular systems, 250 ft^2/ft^3 for spiral wound systems and 2000-5000 ft^2/ft^3 for hollow fine fibers. The systems are in reverse order on the basis of fouling resistance. In general, the tubular system is preferred for dirty water application and the other two for clean water application.

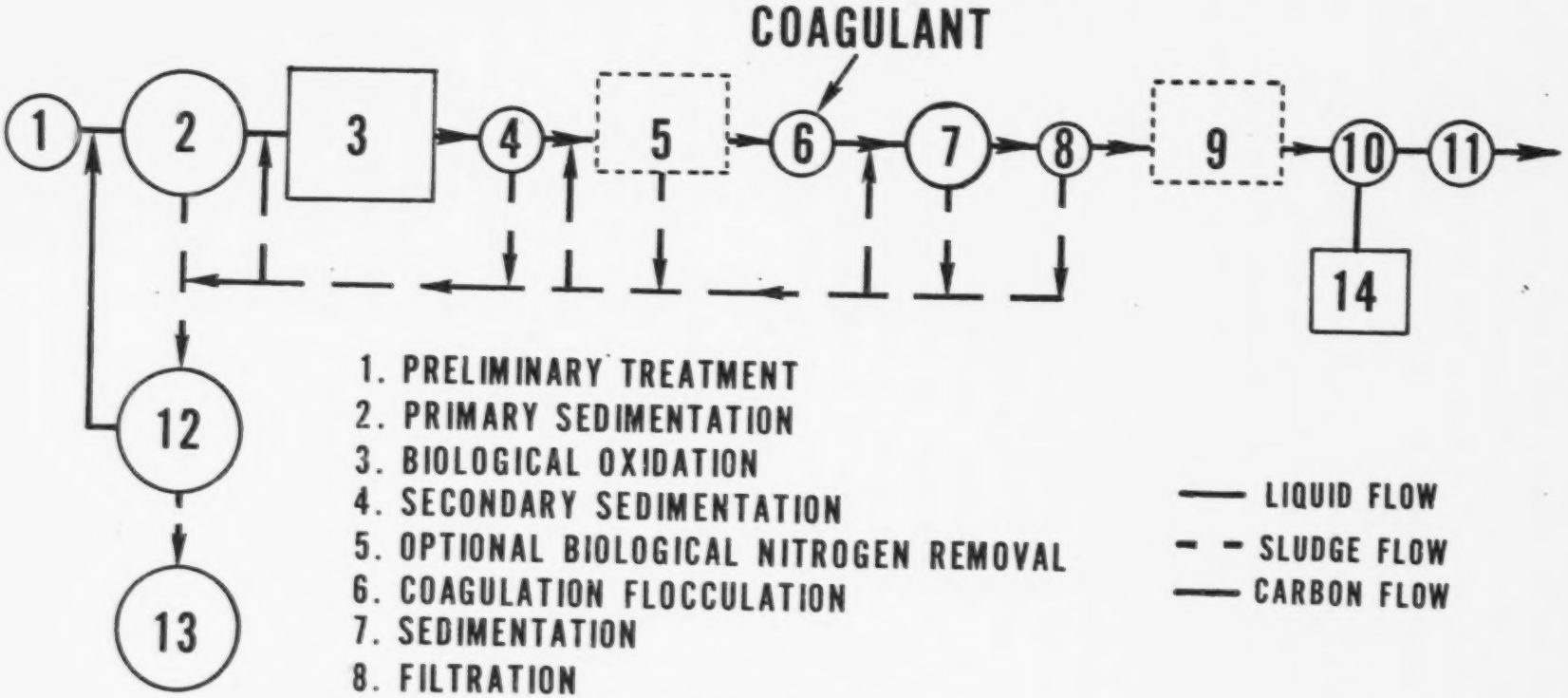
Control of fouling can be achieved by pretreatment, establishment of turbulence at the membrane surface, and chemical cleaning techniques[58]. Application of these techniques is easiest in the tubular system. In addition, tubular systems afford the only opportunity for in-situ membrane replacement. At present none of the systems possess a marked advantage over the others.

The economics of wastewater demineralization is quite imprecise as life times of membranes and resins is not known, nor is degree of fouling and degree of demineralization required. Estimates to demineralize typical domestic sewage range from 20 to 60 cents/1000 gal[56,57,58] without brine disposal costs.

TERTIARY TREATMENT

The term tertiary treatment causes some confusion in that it has a variety of meanings. In the general sense, it represents any stage of treatment or any treatment step applied after secondary treatment. This definition encompasses most of the unit operations and processes discussed in this paper. In a more restricted sense, it represents a specific combination of treatment procedures applied after secondary treatment. In this restricted sense, tertiary treatment is the combination of chemical coagulation, flocculation, sedimentation, deep-bed filtration and activated carbon adsorption. Nitrogen removal procedures are optional in this treatment train. Figure 23 illustrates primary, secondary and tertiary treatment (restricted sense) combined together into a system which provides biological, physical and chemical treatment.

BIOLOGICAL-CHEMICAL-PHYSICAL TREATMENT SYSTEM.
PRIMARY SECONDARY & TERTIARY



1. PRELIMINARY TREATMENT
2. PRIMARY SEDIMENTATION
3. BIOLOGICAL OXIDATION
4. SECONDARY SEDIMENTATION
5. OPTIONAL BIOLOGICAL NITROGEN REMOVAL
6. COAGULATION FLOCCULATION
7. SEDIMENTATION
8. FILTRATION
9. OPTIONAL P-C NITROGEN REMOVAL
10. CARBON ADSORPTION
11. DISINFECTION
12. SLUDGE DEWATERING
13. ULTIMATE SLUDGE DISPOSAL
14. CARBON REGENERATION

FIGURE 23

The ultimate goal of the combination of all these treatment procedures is to produce renovated wastewater, i.e., wastewater which can be reused. A number of installations of plants of this type exist now. These include a 0.5 MGD pilot plant in Nassau County, New York, which has provided feed to test the feasibility of groundwater recharge; a 2 MGD plant at Colorado Springs, Colorado, which provides irrigation flow for public parks and power plant cooling water, and a 7.5 MGD plant at South Lake Tahoe, California, where the effluent is used to provide a recreational lake. The Lake Tahoe plant has been in operation the longest period of time. Table 31 illustrates the performance of this plant at various stages of treatment. These data are indicative of the excellent quality water which can be produced in such a plant. In most respects the quality is superior to that required by the USPHS Standards[27]. Much more testing and evaluation will be required before this water qualifies as a public drinking water supply but it has been approved for fishing and water sports.

Table 32 provides cost estimates for this type of treatment system. These data do not include demineralization costs. A range of values is presented for each plant size to account for various options in the treatment train and because costs are not precisely known for this system.

SUMMARY

In this paper a review of many of the newest developments in wastewater treatment technology have been presented. In the interest of brevity, some areas such as microorganism removal and ultimate disposal have not been covered, and the coverage of others has been less than complete. It should be clear, however, that the wastewater treatment engineer now has at his or her disposal a whole arsenal of treatment procedures. It is well within the realm of practicability to conceive of taking any wastewater and purifying it to any degree desired. All that is necessary is to hook together the various unit operations and processes required. A number of combinations which have already found some utility are summarized in the figures and tables below.

Biological-Physical Treatment

This system employs a filter to upgrade the performance of conventional treatment. It is illustrated in Figure 2.

TABLE 31

WATER QUALITY AT VARIOUS STAGES IN TREATMENT

Parameter	Raw Wastewater	Primary Effluent	Secondary Effluent	Chemical Clarifier Effluent	Filter Effluent	Carbon Effluent	Chlorinated Final Effluent
BOD, mg/l	300	100	30		3	1	0.7
COD, mg/l	280	220	40		25	10	10
SS, mg/l	230	100	26	10	0	0	0
Turbidity, JU	250	150	50	10	0.3	0.3	0.3
MBAS, mg/l	7	6	2.0		0.5	0.10	0.10
Phosphorus, mg/l	12	9	6	2	0.10	0.10	0.10
Coliform (MPN/100 ml)	50 million	15 million	2.5 million		50	50	< 2.0

Biological-Chemical Treatment

In this system, illustrated in Figure 24, chemical coagulants can be added at various points for phosphorus removal and improved solids removal.

Physical-Chemical Treatment

In this system, illustrated in Figure 25, chemical clarification and activated carbon are combined to achieve wastewater purification.

Biological-Physical-Chemical Treatment

This system illustrated in Figure 23 combines a large number of processes to achieve a high degree of purification for a large variety of pollutants. Table 33 summarizes the performance to be expected from these treatment schemes.

Economic Perspective

National average sewerage costs, i.e., collection and treatment, are given in Table 34[59]. It can be seen that the actual cost of treatment is only a minor part of the Nation's wastewater control cost. The cost of treatment is 1.2 cents per day or 12 cents per 1000 gallons. Nationwide adoption of the most complete form of advanced waste treatment (i.e., tertiary with nitrogen removal) would increase treatment costs to about 50¢/1000 gallons or about $18/capita/year. The net effect would be only a doubling of the Nation's wastewater treatment bill because most of the present expense is amortization of the cost of sewage collection. If a lesser degree of treatment were adopted, and it is probable that in most areas water quality standards could be met by a less extensive form of advanced wastewater treatment, the economic impact would be lower. It must also be considered that installation of advanced wastewater treatment will yield significant benefits in environmental improvement and conservation of water resources by providing for at least partial water reuse.

TABLE 32

TREATMENT COST: PHYSICAL-CHEMICAL-BIOLOGICAL SYSTEM

PLANT SIZE, MGD	1	10	100
Cost ¢/1000 gallons	92-120	45-60	25-35

TABLE 33

TREATMENT PERFORMANCE SELECTED WASTEWATER TREATMENT SYSTEMS

Percent Removal

System / Parameter	Conventional	Biological-Physical	Biological Chemical	Physical Chemical	Biological Physical Chemical
Organics	80-90	85-95	90-95	95-99	99+
Suspended Solids	85-95	90-95	95-99	95-99	99+
Phosphorus	0-40	0-50	90-95	95-99	99+

TABLE 34

NATIONAL AVERAGE SEWERAGE COSTS

ACTIVITY	$/Cap/year
Collection (Sewers	13.34
Treatment (Conventional)	4.38
Overhead	2.08
TOTAL	19.80

BIOLOGICAL CHEMICAL TREATMENT SYSTEM

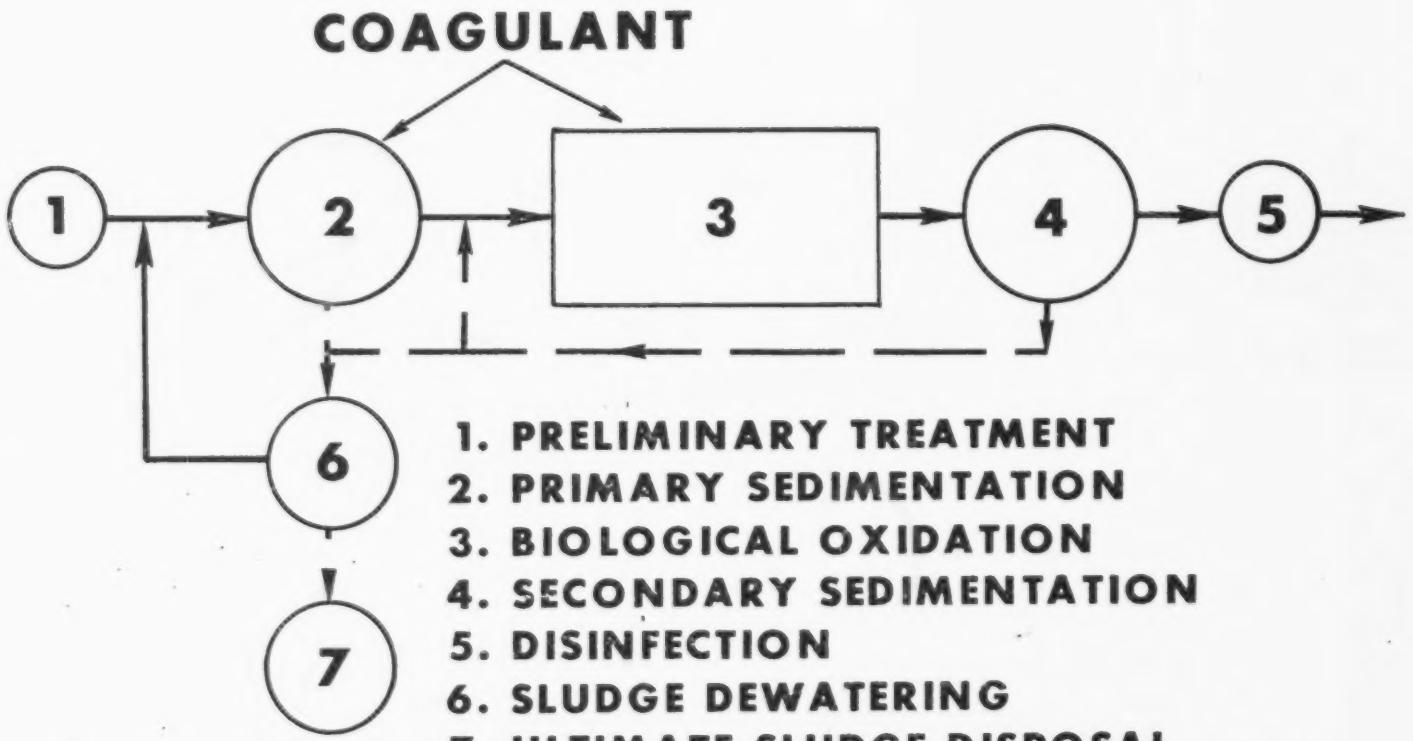
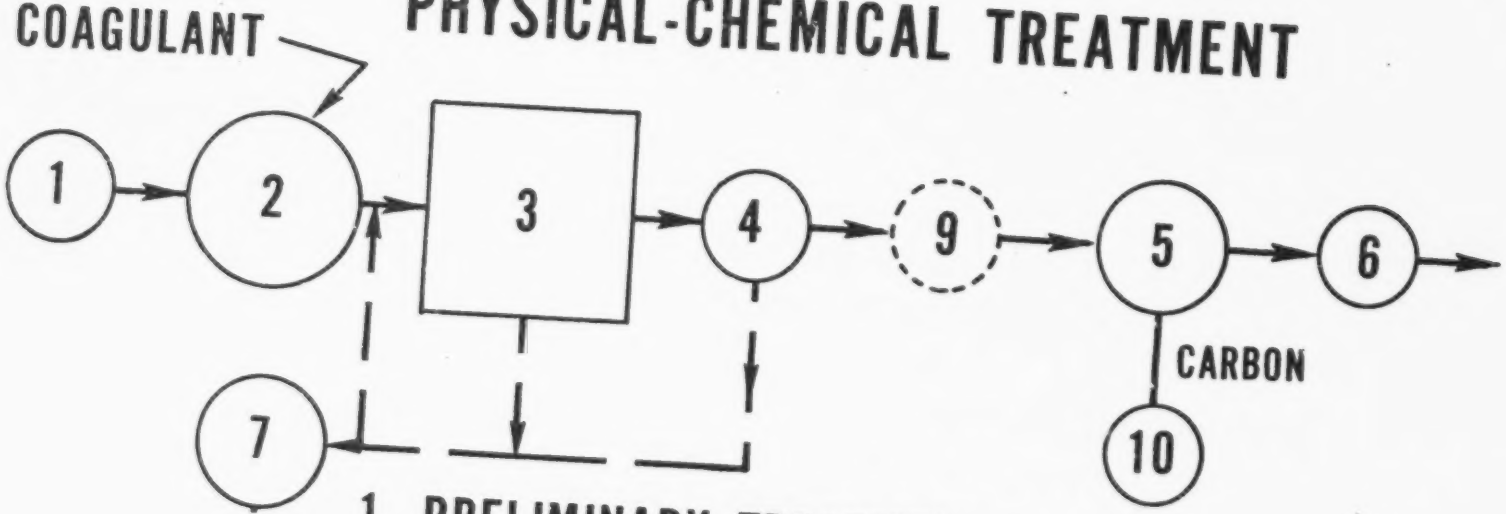


FIGURE 24

PHYSICAL-CHEMICAL TREATMENT



1. PRELIMINARY TREATMENT
2. FLOCCULATION-COAGULATION
3. SEDIMENTATION
4. FILTRATION
5. CARBON ADSORPTION
6. DISINFECTION
7. SLUDGE DEWATERING
8. ULTIMATE DISPOSAL
9. OPTIONAL NITROGEN REMOVAL
10. CARBON REGENERATION

— LIQUID FLOW
- - SLUDGE FLOW
— CARBON FLOW

FIGURE 25

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